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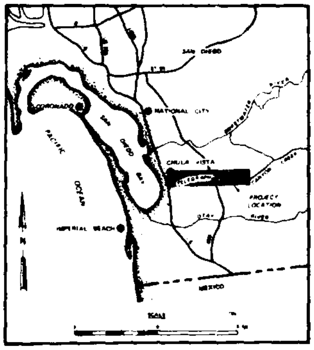

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TECHNICAL REPORT HL-88-26

TELEGRAPH CANYON CREEK CHANNEL IMPROVEMENT PROJECT SAN DIEGO COUNTY, CALIFORNIA

Hydraulic Model Investigation

by

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DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
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Tests were conducted on a 1:20-scale model of Telegraph Canyon to determine the amount of head that would be required to increase the capacity of the culverts from 1,700 cfs to 3,300 cfs, and to develop an energy dissipator design downstream from the existing culverts. Initially, the model reproduced approximately 3,400 ft of the channel and culverts from sta 69+39 to 35+31. Later in the testing program, the model was modified to reproduce upstream channel improvements to sta 78+00 to reflect the change in location of the inlets to the box culverts from sta 59+70 to 67+40.

The required wall heights to contain the additional head that was necessary to increase the capacity of the culverts were determined from water-surface measurements.

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Several inlet designs were tested at the headwall to determine the effect of streamlining the entrance on flow conditions and water-surface elevations. Two of the designs tested performed satisfactorily: a 10-ft-radius curve (type 2 design) and an elliptical shape (type 4 design).

Relocation of the inlet to sta 67+40 had no significant effect on the capacity of the conduits or on the water-surface elevations at the entrance to the conduits. The wall heights in the vicinity of the inlet (sta 67+40) were designed to allow flow to overtop the walls just upstream of sta 68+45 to prevent the surface jump from moving upstream as the discharge increased.

The headwall at the outlet of the existing culverts was skewed, since each of the 84-in.-diam culverts terminated at a different station. This caused unequal distribution of energy in the stilling basin because more flow was entering the left side of the basin. Test results indicated that extending three of the culverts so that all four culverts terminated at a common headwall at sta 39+67 would produce a uniform distribution of flow entering the stilling basin.

The trajectory curve between the outlet and stilling basin as originally designed was 78 ft long. A much shorter (30 ft) and steeper trajectory curve was found to perform just as satisfactorily as the longer and more expensive curve.

The original stilling basin was 60 ft wide and 45 ft long with the apron at elevation (el) -0.08 (elevations are in feet referred to the National Geodetic Vertical Datum). Unsatisfactory flow conditions were observed in this basin with the design discharge and minimum tailwater. A satisfactory design was developed with the 60-ft width by lowering the apron elevation 1 ft and rearranging the basin elements. However, a much more economical design was developed by narrowing the basin width to 40 ft and shortening the apron length to 36 ft. The apron of this basin was at el -4.0, which would require more excavation for construction; but the decrease in size of the basin and transition from the outlet would result in a much less costly structure.

A 24-in.-thick layer of 12-in. (D_{50}) riprap was stable for all discharge and tailwater combinations tested with the recommended stilling basin. The 12-in. riprap was considered the smallest practical size that should be used in the prototype, and smaller sizes were not tested.

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PREFACE

The model investigation reported herein was authorized by the US Army Engineer Division, South Pacific (SPD), on 5 February 1979, at the request of the US Army Engineer District, Los Angeles (SPL). The studies were conducted by personnel of the Hydraulics Laboratory (HL), US Army Engineer Waterways Experiment Station (WES), during the period February 1979 to March 1983. All studies were conducted under the direction of Messrs. H. B. Simmons and F. L. Herrmann, Jr., former and present Chiefs, HL, and J. L. Grace, Jr., Chief of the Hydraulic Structures Division. The tests were conducted by Messrs. J. F. George, D. B. Murray, C. L. Dent, T. E. Murphy, Jr., J. H. Riley, and S. H. Headley II, all of the Locks and Conduits Branch, under the supervision of Mr. G. A. Pickering, Chief of the Locks and Conduits Branch. This report was prepared by Mr. Pickering and Mr. George and edited by Mrs. Marsha Gay, Information Technology Laboratory.

Prior to design and construction of the model, Messrs. Murray and Pickering visited the project site to inspect the existing culverts. Messrs. S. B. Powell, Headquarters, US Army Corps of Engineers; Ted Albrecht, SPD; and Bob Koplin, K. L. Wanner, and Phil Tryou, SPL, visited WES during the study to discuss test results and to correlate these results with concurrent design work.

COL Dwayne G. Lee, EN, is the Commander and Director of WES. Dr. Robert W. Whalin is the Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.4535924	kilograms
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	millimetres
miles (US statute)	1.609344	kilometres
pounds	0.4535924	kilograms
square miles	1.609344	square kilometres

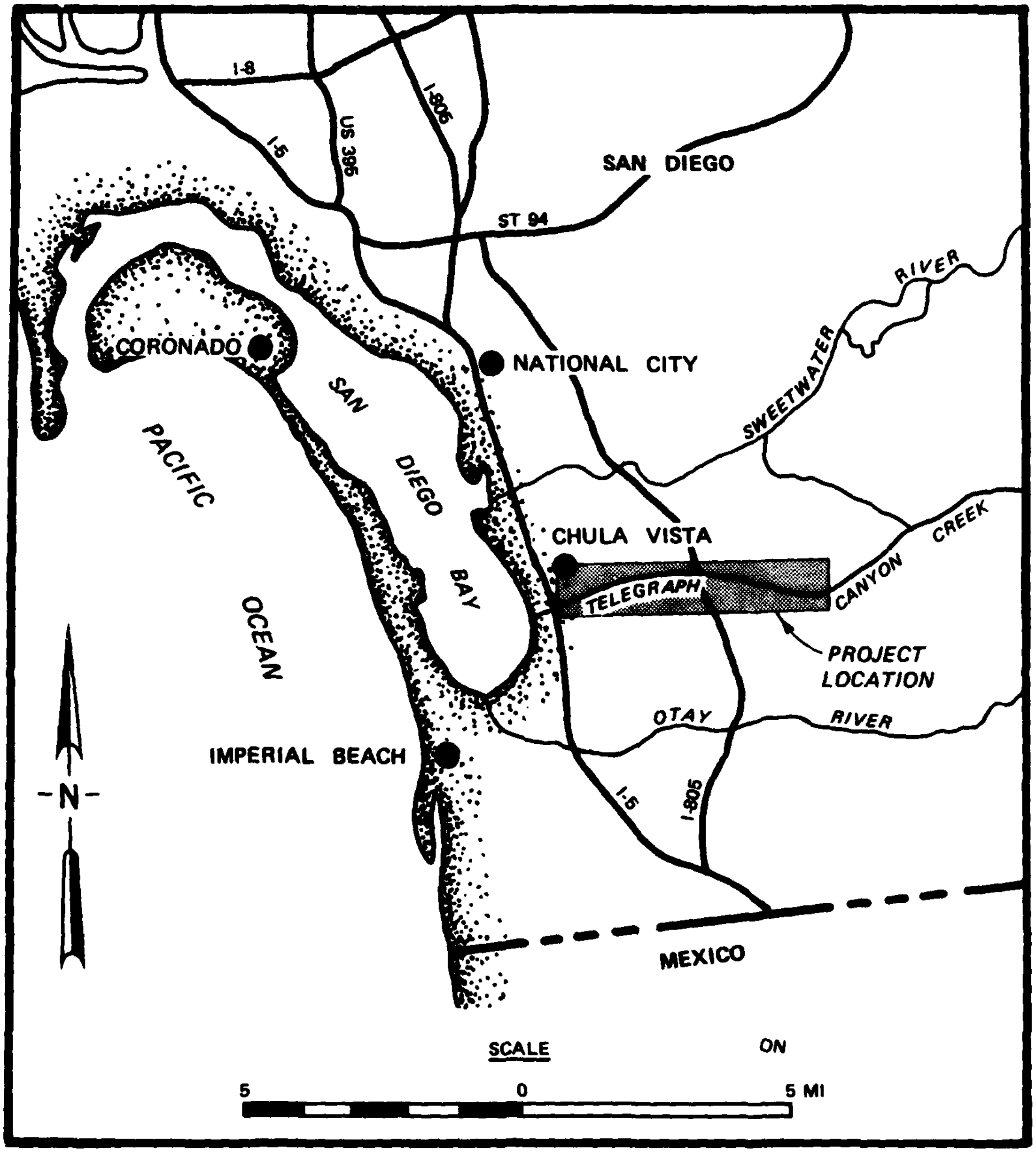

Figure 1. Vicinity map

TELEGRAPH CANYON CREEK CHANNEL IMPROVEMENT PROJECT

SAN DIEGO COUNTY, CALIFORNIA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Telegraph Canyon Creek originates in the hills east of the city of Chula Vista, California, and flows in a westerly direction through Chula Vista into the southern portion of San Diego Bay (Figure 1). The basin is long and narrow, the average width being less than 1 mile* and the length about 10 miles; the total drainage area is about 7.5 square miles. Because climatic and drainage area characteristics are not conducive to continuous flow, little streamflow occurs except during and immediately following rains. Runoff increases rapidly in response to rainfall excess.

2. Existing culverts that convey flow from Telegraph Canyon Creek underneath Interstate 5 and other nearby streets consist of four 84-in.-diam pipes approximately 150 ft long that transition into three 8-ft-wide by 7-ft-high box culverts approximately 1,000 ft long that transition back into four 84-in.-diam pipes that are approximately 100 ft long (Plate 1). Capacity of this system is 1,700 cfs.

3. The proposed plan of improvement consists of approximately 750 ft of double box culvert, 12 ft wide and 10 ft high; about 4,200 ft of rectangular concrete channel; and an inlet structure about 300 ft long upstream from the concrete channel. An energy dissipator and approximately 2,000 ft of trapezoidal earth-bottom channel will be provided downstream from the existing culverts. The proposed channel would convey the future 100-year flood having a peak discharge of 3,300 cfs from the inlet structure to San Diego Bay.

Purpose and Scope of Model Investigation

4. The purpose of the model investigation was to determine the adequacy

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* Non-SI units of measurement used in this report can be converted to SI (metric) units as shown on page 3.

of and develop desirable modifications to the proposed channel and conduits and existing conduits so that the improvement project would contain the 100-year design flood.

5. Because energy losses and changes in flow distribution in the proposed conduit system could not be determined by analytical means, a model study was needed to determine the following:

- a. The amount of head that would be required to increase the capacity of the existing culverts from 1,700 cfs to 3,300 cfs (design of the upstream channel walls would then be based on this head).
- b. Flow conditions throughout the proposed channel and in the proposed and existing conduits.
- c. An energy dissipator design downstream from the culverts.

PART II: THE MODEL

Description

6. The model, constructed to a scale of 1:20, initially reproduced approximately 3,400 ft of the channel and culverts from sta 69+39 to 35+31 (Figure 2, Plate 1). The high-velocity channel, culverts, and energy dissipator were constructed of transparent plastic so that flow conditions could be observed. Initially, the downstream channel was molded in sand and cement mortar to sheet metal templates to test the energy dissipator and measure velocities. In later tests, the cement mortar was replaced with riprap and sand to determine the optimum protection plan downstream from the energy dissipator.

Model Appurtenances

7. Water used in the operation of the model was supplied by a circulating system. Discharges were measured with venturi meters installed in the flow lines and were baffled before entering the model. Velocities were measured with pitot tubes that were mounted to permit measurement of flow from any direction and at any depth. Water-surface elevations were measured with point gages, and pressures were measured with piezometers. Different designs and various flow conditions were recorded photographically.

Scale Relations

8. The accepted equations of hydraulic similitude, based on the Froude criteria, were used to express mathematical relations between the dimensions and hydraulic quantities of the model and prototype. General relations for the transference of model data to prototype equivalents are presented in the following tabulation. Model measurements of discharge, water-surface elevations, and velocities can be transferred quantitatively to prototype by the scale relations. Experimental data also indicate that the model-to-prototype scale ratio is valid for scaling riprap in the sizes used in this investigation.

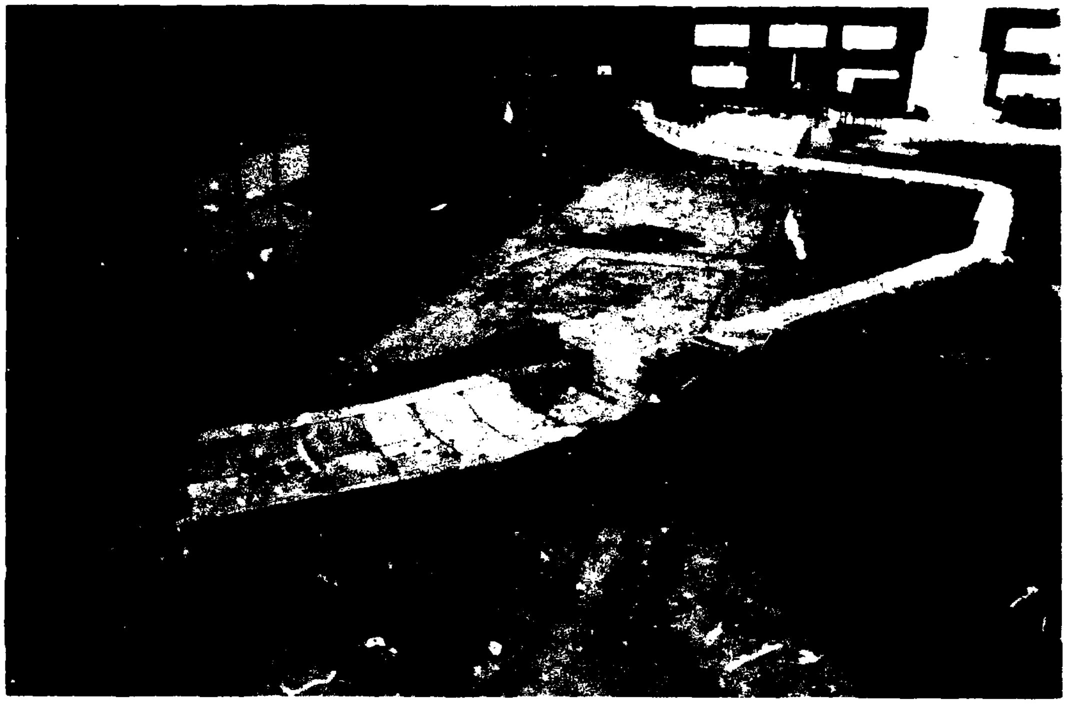


Figure 2. General view of model

Characteristic	Dimension*	Scale Relations Model:Prototype
Length	L_r	1:20
Area	$A_r = L_r^2$	1:400
Velocity	$V_r = L_r^{1/2}$	1:4.472
Time	$T_r = L_r^{1/2}$	1:4.472
Discharge	$Q_r = L_r^{5/2}$	1:1,789
Weight	$W_r = L_r^3$	1:8,000
Roughness coefficient	$N_r = L_r^{1/6}$	1:1.648

* Dimensions are in terms of length.

Model Adjustments

9. The coefficient of roughness of the model of the high-velocity channel had previously been determined from previous test data to be approximately

0.009 (Manning's n). Basing similitude on the Froude relation, this n value would be equivalent to a prototype n of 0.0148. The n value used in the design of the prototype channel was 0.014; therefore supplementary slopes were added to the model channel to correct for this difference in the n value of the model and prototype.

10. Making a valid study of the flow conditions in a closed conduit flowing full required that the hydraulic grade line be simulated accurately in the model. It is not possible to satisfy the requirements of both the Froude and Reynolds criteria for complete similitude by using water in the model. Since water is also the fluid in the prototype and hydraulic similitude between the model and prototype was based on Froude relations, the Reynolds number with the design flow in the model was lower than that in the prototype. Therefore, the resistance coefficient of the model was higher than that of the prototype, and the excess losses in the model were compensated for by shortening the length of the model culverts.

PART III: TESTS AND RESULTS

11. Tests were conducted to observe general flow conditions, determine the required channel wall heights upstream from the closed conduits, develop an optimum energy dissipator downstream from the closed conduits, and determine the riprap requirements downstream from the energy dissipator.

Open Rectangular Channel

Inlet at sta 59+70

12. The existing culverts under Interstate 5 have a capacity of 1,700 cfs with existing upstream conditions. In the plan of improvement, these culverts will be used to convey the design discharge of 3,300 cfs by increasing the head upstream. Thus, the walls of the open rectangular channel (Plate 1) will have to contain the buildup of flow required to create this head. A headwall will be required at the entrance to the closed culverts at sta 50+70 (Figures 3 and 4). Several inlet designs were tested at the headwall in an effort to reduce the required head as much as practical while retaining good flow conditions.

13. Type 1 inlet. The type 1 inlet consisted of a 3-ft-radius curve at the headwall with a sloping splitter wall between the two culverts slightly downstream from the headwall as shown in Plate 2. With the design discharge, flow conditions were unstable at the inlet with intermittent vortices (Photo 1) present along both walls of the rectangular channel. A hydraulic jump occurred upstream around sta 66+14; however, it was not stationary and tended to move back and forth about 50 ft upstream and downstream from this station. Water-surface profiles with the type 1 inlet design are shown in Plate 3.

14. Type 2 inlet. For the type 2 inlet (Plate 2), the radius at the headwall was increased to 10 ft, and the splitter wall was extended upstream to sta 59+70 to further streamline the flow entering the closed conduit. The splitter wall had a rounded nose with a radius of 0.5 ft. With this design, the hydraulic jump occurred a little farther downstream in the open channel with the design discharge as shown in Plate 4. Flow conditions at the inlet (Photo 2) were more stable than those of the original inlet design.

15. Type 3 inlet. The splitter wall was extended 13.5 ft upstream of

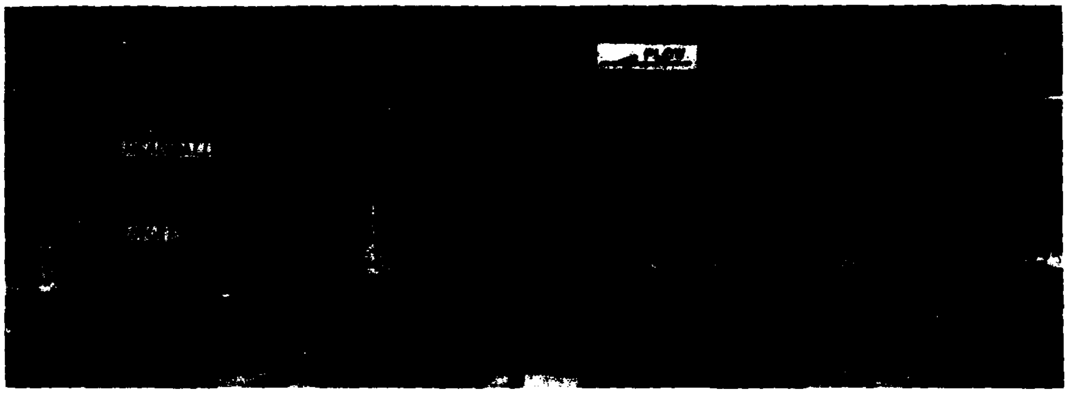


Figure 3. Open channel and headwall upstream from culverts

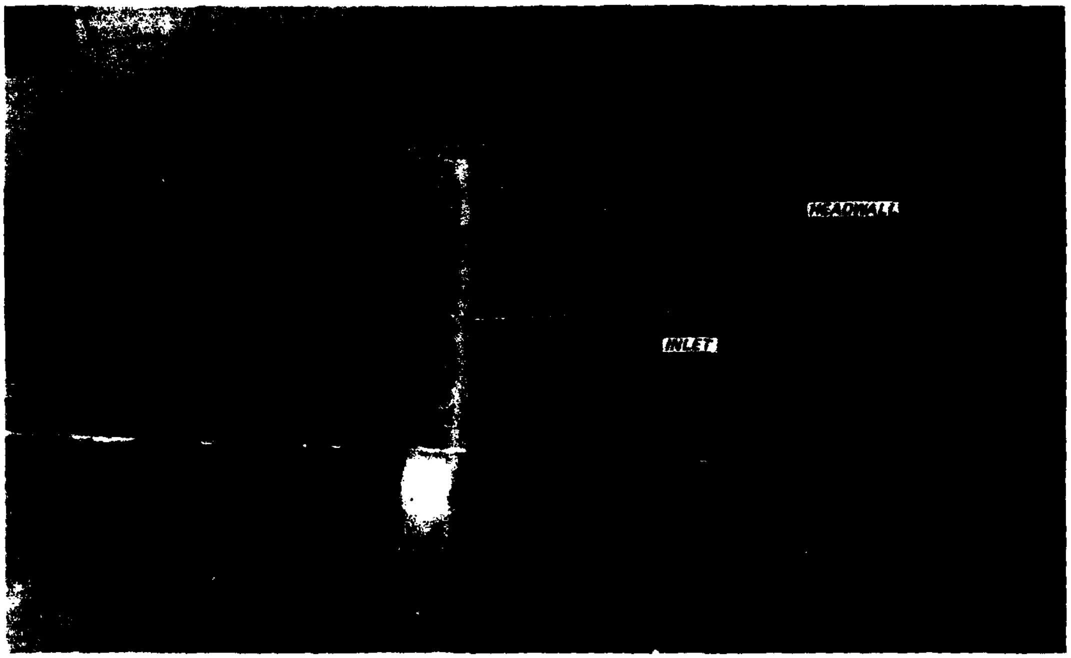


Figure 4. Inlet at sta 59+70

sta 59+70 in the type 3 inlet (Plate 2). This had no noticeable effect on water-surface profiles and flow conditions. Thus, the extended wall was removed from the model, since it would add an unnecessary expense in construction of the prototype.

16. Type 4 inlet. An elliptical curve, rather than the 10-ft-radius curve, was placed at the headwall in the type 4 design (Plate 2). This had little effect on water-surface profiles upstream, and the sponsor estimated that it would be more expensive to construct in the prototype.

17. Recommended design inlet. Comparisons of water-surface profiles in the open channel and observations of flow conditions at the inlet indicated the type 2 inlet to be the optimum design tested. Thus, it was recommended for the prototype.

18. Tests conducted with the type 2 inlet with and without the 3-ft-high low-flow training walls in the existing transitions in the culverts downstream indicated that the presence of these walls has no effect on the discharge capacity of the structure. These training walls were constructed in the prototype to deflect debris so that it would pass through the system instead of causing silting problems at low flows.

19. Water-surface profiles were measured in the open channel with discharges of 2,800 and 3,500 cfs in addition to those measured with the design discharge. These data, shown in Plate 4, can be used to estimate the location of the hydraulic jump in the open channel when the water surface reaches the top of the walls at the culvert entrance.

20. Approximately 750 ft of double box culvert (Figure 5), 12 ft wide by 10 ft high, will be constructed upstream from the existing culverts (Figure 6). Details of the proposed and existing culverts are shown in Plates 5-7.

21. Various flows were observed in the model to determine if any problems existed with partial conduit flow and to determine at what discharge the conduits would initially flow full. With discharges up to 1,700 cfs, partial conduit flow was present throughout the conduit system and no problems were observed. At a discharge of 1,700 cfs, the four-barrel conduit between sta 52+16 and 50+92 began to flow full with partial flow conditions still present in the rest of the conduit system. When the flow was gradually increased to approximately 2,400 cfs, the majority of the conduits flowed full with a hydraulic jump present at the inlet (sta 59+70). The entire conduit

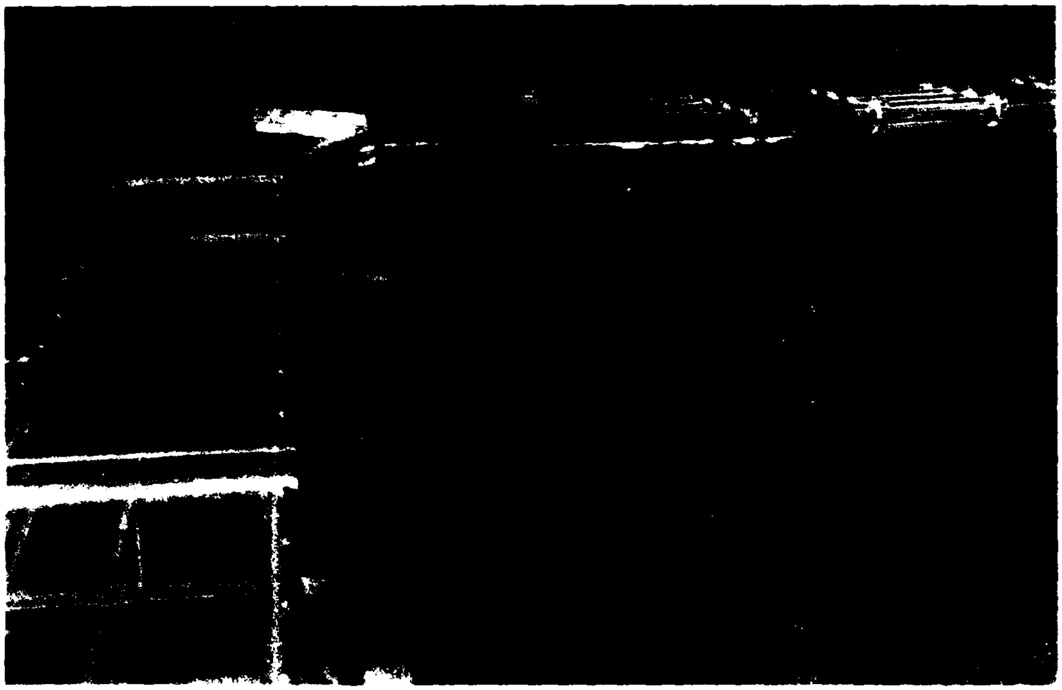


Figure 5. New double box culvert and transition into existing structure

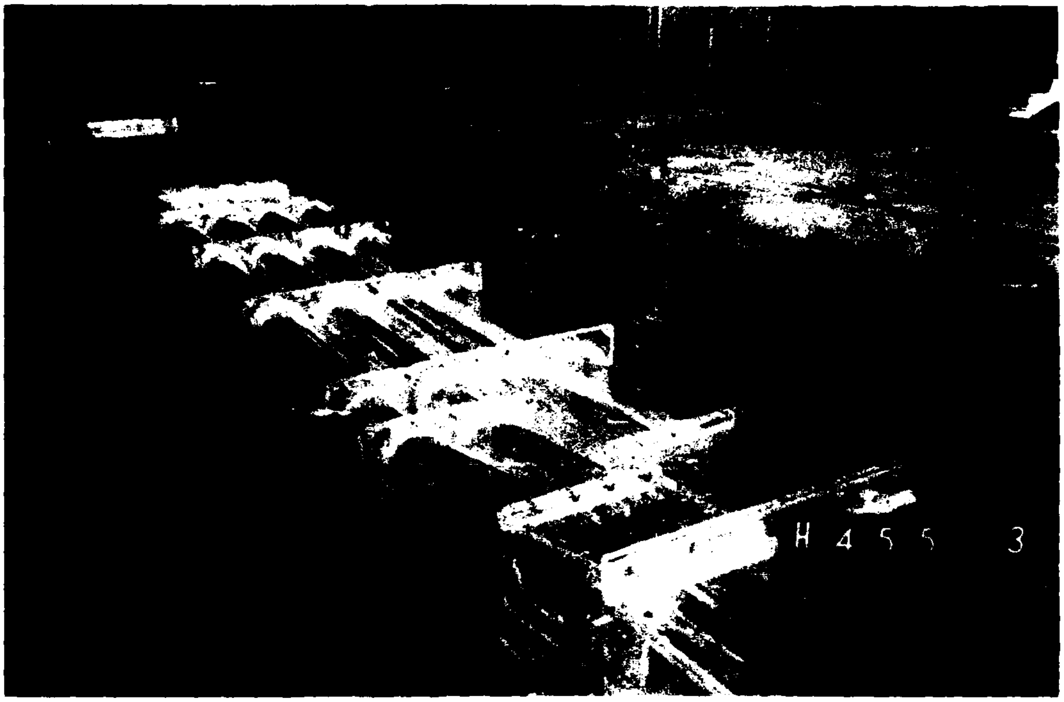


Figure 6. Existing culverts and proposed transition from double box culvert

system eventually flowed full with a discharge of approximately 2,800 cfs.

22. With the design discharge of 3,300 cfs, pressures were measured at several locations on the roof of the conduits to determine the head losses in the conduits between sta 54+18 and 50+51. Pressures acting on the roof of the existing culverts reached a maximum of approximately 8 ft of water during the design flow. Pressures in the proposed double box culvert reached a maximum of approximately 10 ft of water at the entrance and at the transition to the four 84-in.-diam pipes for the same design flow. These maximum pressures are listed in the following tabulation. Locations of the piezometers are shown in Plate 8.

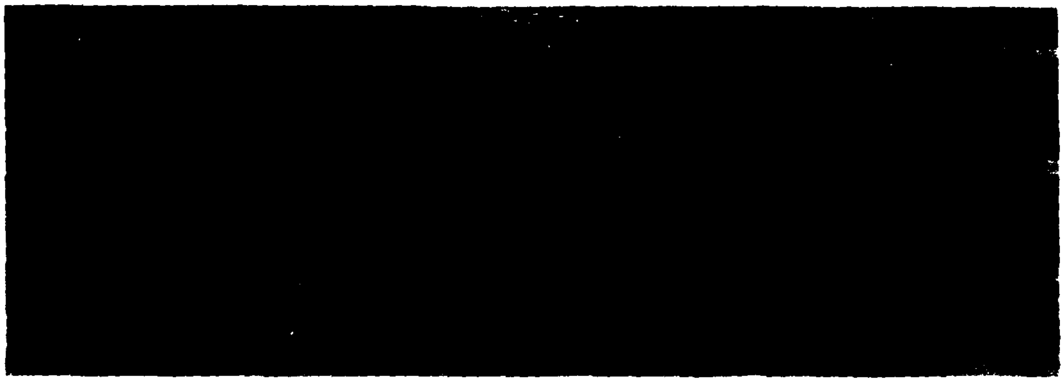
Sta	Piezometer No.	Piezometer Elevation*	Pressure Elevation*
54+18	1	30.72	41.0
	2	30.72	40.0
52+78	3	30.29	40.3
	4	30.29	40.2
52+10	5	27.06	34.6
	6	27.06	34.7
	7	27.06	34.9
	8	27.06	34.6
50+51	9	26.12	32.7
	10	26.12	32.9
	11	26.12	32.8

* All elevations (el) cited in this report are in feet referred to the National Geodetic Vertical Datum (NGVD).

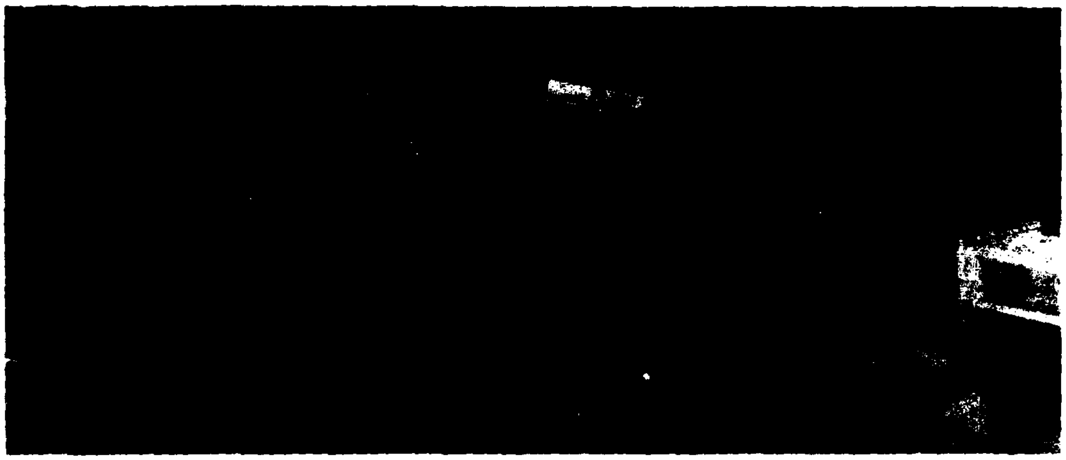
Inlet at sta 67+40

23. The upstream portion of the model was modified as requested by the sponsor to reproduce two 10- by 13-ft conduits that replaced the open channel section from sta 59+70 to sta 67+40 (new location of the box culvert inlet). The open channel was extended upstream from sta 69+39 to sta 78+00 in the model. The conduit and channel modifications are shown in Figure 7. The 861 ft of open channel reproduced contained superelevated curves that were rotated about the center line. The channel had a uniform width of 18 ft from sta 78+00 to sta 68+60 which transitioned to 27 ft at sta 67+70. The inlet into the double-barrel conduit located just downstream of the transition contained the type 4 inlet design previously tested. After more detailed study,

a. Open channel section between sta 78+00 and 70+50



b. Transition section in open channel between sta 69+00 and 67+40 and double-barrel conduit between sta 67+40 and 63+00



c. Double-barrel conduit beteen sta 63+00 and sta 58+25

Figure 7. Dry beds of channel and conduit modifications

the sponsor determined that this design was less costly to construct than the type 2 design that was recommended from earlier tests. These tests had also indicated that the hydraulic performance of the two designs was the same. The initial model wall heights in the open channel were constructed high enough to prevent overtopping at the design discharge. Details of the channel and conduit extension are shown in Plates 9 and 10. A general view of the model with the channel extended upstream to sta 78+00 is shown in Figure 8.

24. General flow conditions observed throughout the open channel and double-barrel conduit were found to be satisfactory with discharges ranging from 500 cfs to 3,600 cfs (300 cfs above design flow) with the normal depth set at the entrance to the model for each discharge. Partially full flow conditions were observed in the double-barrel conduit with discharges less than 2,600 cfs. With a discharge of 2,600 cfs, the conduit in the vicinity of the inlet began to flow full with the toe of the hydraulic jump at sta 67+80 (Photo 3a). When the discharge was increased to 2,800 cfs, fairly stable flow conditions were present in the transition as shown in Photo 3b. At a discharge of 3,300 cfs, the hydraulic jump became more of a surface jump located at the upstream end of the transition with intermittent vortex action present at the headwall at sta 67+40 (Photo 3c). As the discharge was increased to 3,600 cfs, the surface jump moved upstream to approximately sta 76+00. Discharges greater than 3,600 cfs were not observed because the surface jump would continue to move upstream and exceed the model limits. Water-surface profiles for discharges of 2,800 cfs, 3,300 cfs, and 3,600 cfs are provided in Plates 11-13, respectively.

25. Proposed wall heights furnished by the sponsor were then installed in the model between sta 70+50 and 67+40. The wall heights were designed to contain the flow from sta 68+45 to sta 67+40, but allow flow to overtop the walls just upstream of sta 68+45 to prevent the surface jump from moving upstream as the discharge increases. The overtopping flow would spill over into a small bypass channel and be diverted to another area.

26. Initial tests indicated that the water surface exceeded the proposed wall heights between sta 68+45 and sta 67+40 for the higher discharges. The wall heights were increased in this area to prevent overtopping previously observed. Water-surface profiles were recorded with discharges of 3,300 cfs, 3,400 cfs, 3,500 cfs, and 4,000 cfs, as shown in Plates 14-17, respectively.

Figure 8. Looking upstream at general view of model with channel extended to sta 78+00

Flow conditions with the increased wall heights installed are shown in Photo 4.

Stilling Basin

Type 1 (original) design

27. The original design stilling basin (Figure 9) was 60 ft wide and 45 ft long. The basin apron was at el -0.08, and the basin contained two rows of 2-ft-high baffle blocks. The basin was connected to the culvert outlet with a curved trajectory 78 ft long. The headwall at the outlet was skewed since the four 84-in.-diam culverts terminated at different locations. Details of the original design stilling basin are shown in Plate 18.

28. Tests were conducted with the type 1 stilling basin to observe the hydraulic performance with discharges up to 3,300 cfs. With the design discharge of 3,300 cfs and minimum expected tailwater elevation of 8.85, unsatisfactory energy dissipation was observed in the basin. The hydraulic jump occurred downstream from the toe of the trajectory (Photo 5) due to the minimum tailwater depth, resulting in high velocities in the downstream exit channel. Also, the staggered culvert outlets created unequal flow distribution at the toe of the trajectory at sta 38+56, resulting in higher flows along the left side of the stilling basin. Velocities measured at the end of the basin are shown in Plate 19.

Alternate designs

29. Extensions were added to three of the existing culverts, resulting in all the culvert outlets terminating at sta 39+67. This produced equal flow distribution on the trajectory and in the stilling basin. However, the hydraulic jump was still downstream from the toe of the trajectory with the design discharge and minimum tailwater.

30. The basin apron was lowered 1 ft to el -1.08 to increase the tailwater depth available to maintain a stable hydraulic jump with the design discharge and minimum tailwater. The baffle blocks were also moved 5 ft closer to the toe of the trajectory and the basin was shortened to 36 ft. Details of this design (type 2) are shown in Plate 20. Satisfactory flow conditions were observed throughout the stilling basin and exit channel for discharges up to 3,300 cfs. Water-surface profiles in the stilling basin are shown in Plate 21.

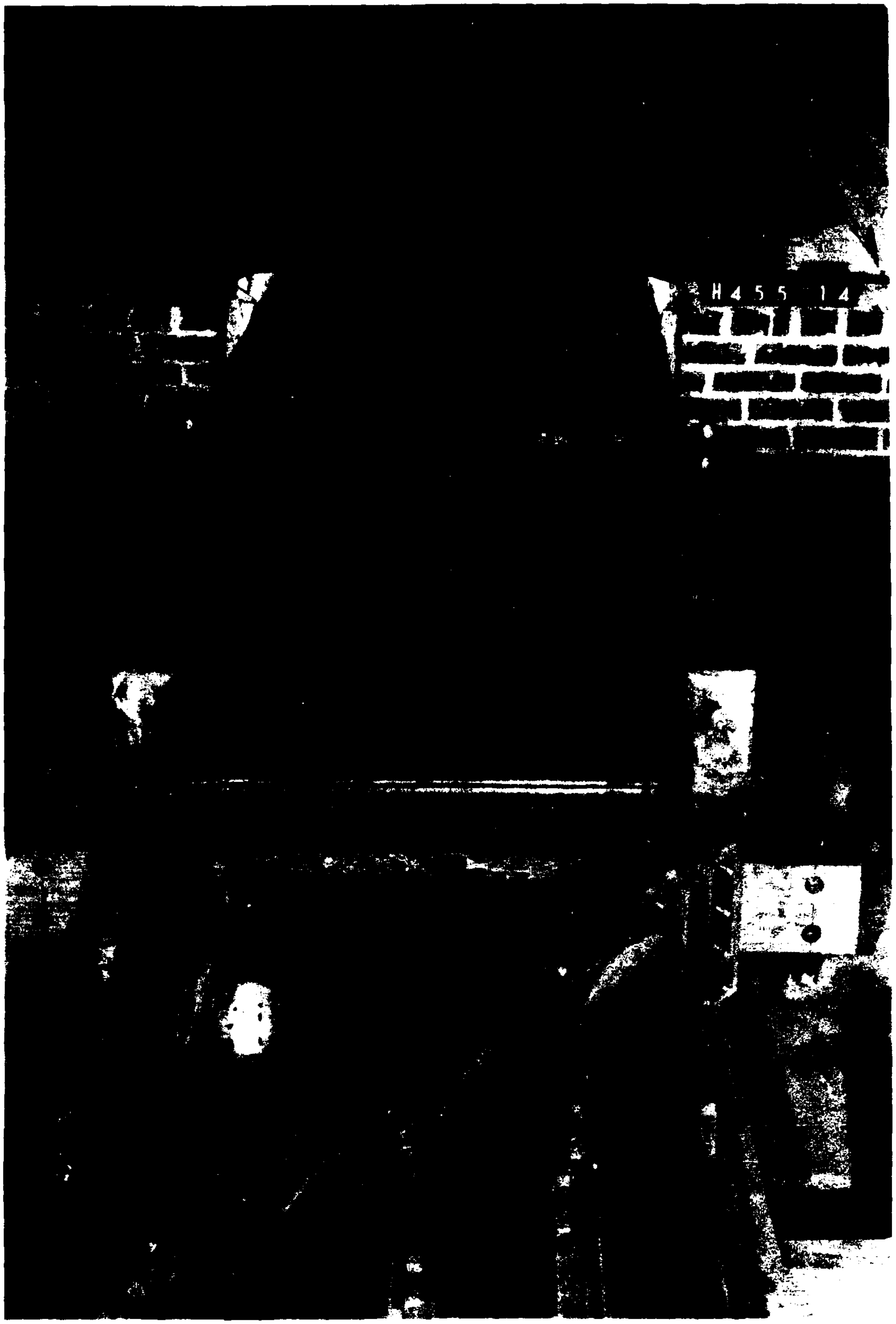


Figure 9. Type 1 (original) stilling basin

31. Although the type 2 design basin performed satisfactorily, the width of the basin was reduced from 60 to 40 ft in an effort to reduce the cost of the structure. The apron was maintained at el -0.08 initially. Tests were conducted with basin lengths ranging from 21 to 39 ft and different arrangements of 3-ft-high baffle blocks. None of these basins would maintain a hydraulic jump within the basin with the minimum tailwater, el 8.85.

32. Tests were conducted to determine the tailwater elevation required to maintain a good jump within the basin with the apron at el -0.08. The tests consisted of adjusting the tailwater elevation until the tailwater depth was adequate to produce a satisfactory hydraulic jump in the stilling basin. The results from these tests gave a good indication of how much the apron needed to be lowered to obtain a good hydraulic jump with minimum tailwater conditions. The adjusted tailwater elevation was approximately 4 ft higher than the minimum expected at the project; therefore, the basin apron was lowered to el -4.0. Also, a much steeper and shorter trajectory was used to connect the outlet with the stilling basin. This trajectory shape was based on the theoretical equation for a free trajectory using a velocity of 30 fps, which was 1.25 times the average flow velocity measured at the beginning of the curve. Piezometers located on the trajectory indicated that no negative pressure would occur with the design discharge of 3,300 cfs. Minimum pressures measured on the center line of the stilling basin trajectory with the design discharge are listed in the following tabulation. Performance of the basin was evaluated with various basin lengths and basin element arrangements.

Piezometer No.	Piezometer El	Pressure El
1	12.5	14.0
2	12.0	13.0
3	11.5	12.6
4	11.0	12.1
5	10.0	10.9
6	8.0	9.1
7	5.0	5.7
8	3.0	5.1

Note: $y = 0.0058X - 0.0179X^2$.

Figure 10. Recommended stilling basin (type 10) with
type 2 riprap plan

Recommended design

33. The recommended stilling basin design (Figure 10), type 10, consisted of a 36-ft-long apron with one row of 2-ft-high baffle blocks placed 18 ft downstream from the toe of the trajectory and a 2-ft-high, 1V on 1H sloping end sill. The basin was 40 ft wide and was connected to the outlet by a 30.4-ft-long trajectory. Details of this design are shown in Plate 22. The basin provided satisfactory flow conditions throughout the stilling basin and exit channel for the full range of discharges and tailwater elevations. Flow conditions with discharges of 1,000, 2,000, and 3,300 cfs are shown in Photo 6. Water-surface profiles and velocities measured in the stilling basin and exit channel are shown in Plate 23.

Riprap Protection

34. Using the type 10 design stilling basin, different riprap schemes were tested to determine the minimum size and extent of exit channel protection required with the design discharge. The width of the exit channel at the

end sill was increased 5 ft on each side of the stilling basin and the 1V on 2H side slopes in the exit channel were extended and curved back to the stilling basin walls (Plate 22). This provided an area for the flow leaving the stilling basin to expand and dissipate kinetic energy effectively in turbulence rather than in direct attack on the side slopes.

Type 1 riprap plan

35. The type 1 riprap plan (Plate 22) consisted of riprap with an average diameter D_{50} of 12 in. and a blanket thickness of 24 in. The riprap extended downstream to sta 37+50, a distance of approximately 118 ft. The gradation of the riprap tested in the model is shown in Plate 24. Tests were conducted with flow ranging from 1,000 to 3,300 cfs, with each test lasting a minimum of 10 hr (prototype). No failure was observed after these tests. The riprap used in these tests represents the minimum size riprap deemed practical for the prototype; therefore, smaller size riprap was not tested in the model.

Type 2 riprap plan

36. Additional tests were conducted to determine performance with less riprap protection in the exit channel. The length of 12-in. D_{50} riprap protection downstream from the stilling basin was reduced approximately 48 ft to sta 37+97.6 (type 2 riprap plan, Figure 10 and Plate 25). Tests were conducted with various discharges up to 3,300 cfs, and each test simulated a minimum duration of 10 hr (prototype). No failure was observed after these tests. The type 2 riprap plan provided adequate protection immediately downstream from the stilling basin.

37. A shorter length of riprap was not tested because this would have steepened the invert adverse slope and shortened the length of transition of the side slopes. This overall change in the configuration of the channel immediately downstream of the stilling basin would have increased the velocities in the exit channel. An increase in the exit velocities was noted when comparing types 1 and 2 riprap protection plans (Plates 23 and 26, respectively).

PART IV: DISCUSSION OF RESULTS AND RECOMMENDATIONS

38. Model tests indicated that the existing culverts and transitions underneath Interstate 5 would carry a 100-year frequency flood of 3,300 cfs if the walls of the upstream open channel were high enough to contain the buildup of flow. The required wall heights were determined by measuring water-surface elevations in this area. As the buildup of flow occurred at the headwall entrance to the closed culvert, a hydraulic jump formed in the channel, since flow from the upstream channel was supercritical. The location of this jump was very sensitive to the elevation of the water surface at the headwall, and only a slight change in water-surface elevation resulted in the location of the jump moving upstream or downstream by as much as 50 ft. Thus, data were obtained with discharges other than the design discharge to estimate the location of the hydraulic jump when water-surface elevations reached the top of the proposed walls.

39. Several inlet designs were tested at the headwall to determine the effect of streamlining the entrance on flow conditions and water-surface elevations. Two of the designs tested performed satisfactorily: a 10-ft-radius curve (type 2 design) and an elliptical shape (type 4 design). Construction costs should determine which of these inlet designs is used in the prototype.

40. Initially, the inlet designs tested were located at sta 59+70. Later in the testing program the model was modified, as requested by the sponsor, to extend the box culverts upstream to sta 67+40. There was no significant effect on the capacity of the conduits or the water-surface elevations at the entrance to the conduit. The wall heights in the vicinity of the inlet were designed to contain the flow from sta 67+45 to 67+40, but allow the flow to overtop the walls just upstream of sta 68+45, to prevent the surface jump from moving upstream as the discharge increased. The overtopping flow spilled over into a small bypass channel and was diverted to another area.

41. In the existing culverts there is a transition from four 84-in.-diam pipes to a triple box and a transition back to four 84-in.-diam pipes. Several interior walls in these transitions were catching debris in the prototype and in turn causing silting with low flows. Three-foot-high training walls were constructed in the prototype transitions to deflect debris so that it would pass through the system. Tests conducted both with and without these

walls in the model indicated that they would have no effect on the discharge capacity of the system.

42. Pressures acting on the roof of the existing culverts reached a maximum of approximately 8 ft of water during the design flow. Pressures in the proposed double box culvert reached a maximum of approximately 10 ft of water at the entrance and at the transition to the four 84-in.-diam pipes, for the same design flow.

43. The headwall at the outlet of the existing culverts is skewed, since each of the 84-in.-diam culverts terminates at a different station. This causes unequal distribution of energy in the stilling basin because more flow is entering the left side of the basin. Three of the culverts should be extended so that they will terminate at a common headwall at sta 39+67.

44. The trajectory curve between the outlet and stilling basin as originally designed was 78 ft long. This trajectory performed satisfactorily, but was unnecessarily long. A much shorter (30 ft) and steeper trajectory curve was found to perform just as satisfactorily as the longer and more expensive curve. This trajectory was based on the theoretical equation for a free jet using a velocity 1.25 times the average velocity measured at the beginning of the curve with the design discharge. No negative pressures were measured on the trajectory.

45. The original stilling basin was 60 ft wide and 45 ft long with the apron at el -0.08. Unsatisfactory flow conditions were observed in this basin with the design discharge and minimum tailwater. A satisfactory design was developed using the 60-ft width, lowering the apron elevation 1 ft, and rearranging the basin elements. However, a much more economical design was developed by narrowing the basin width to 40 ft and shortening the apron length to 36 ft. The apron of this basin was at el -4.0, which would require more excavation for construction; but the decrease in size of the basin and transition from the outlet would result in a much less costly structure. This structure performed satisfactorily throughout the expected range of discharges and tailwaters.

46. A 24-in.-thick layer of 12-in. D_{50} riprap was stable for all discharge and tailwater combinations tested with the recommended stilling basin. The 12-in. riprap was considered the smallest practical size that should be used in the prototype and smaller sizes were not tested. Two riprap protection plans were tested downstream from the recommended stilling basin. One

extended a distance of approximately 120 ft downstream from the basin, and the other one extended only 70 ft downstream. Either plan was adequate for protecting the area immediately downstream from the structure. If a good vegetative cover can be established in the natural earth channel downstream from the riprap, the shorter plan should be adequate; if not, the longer plan should be used.

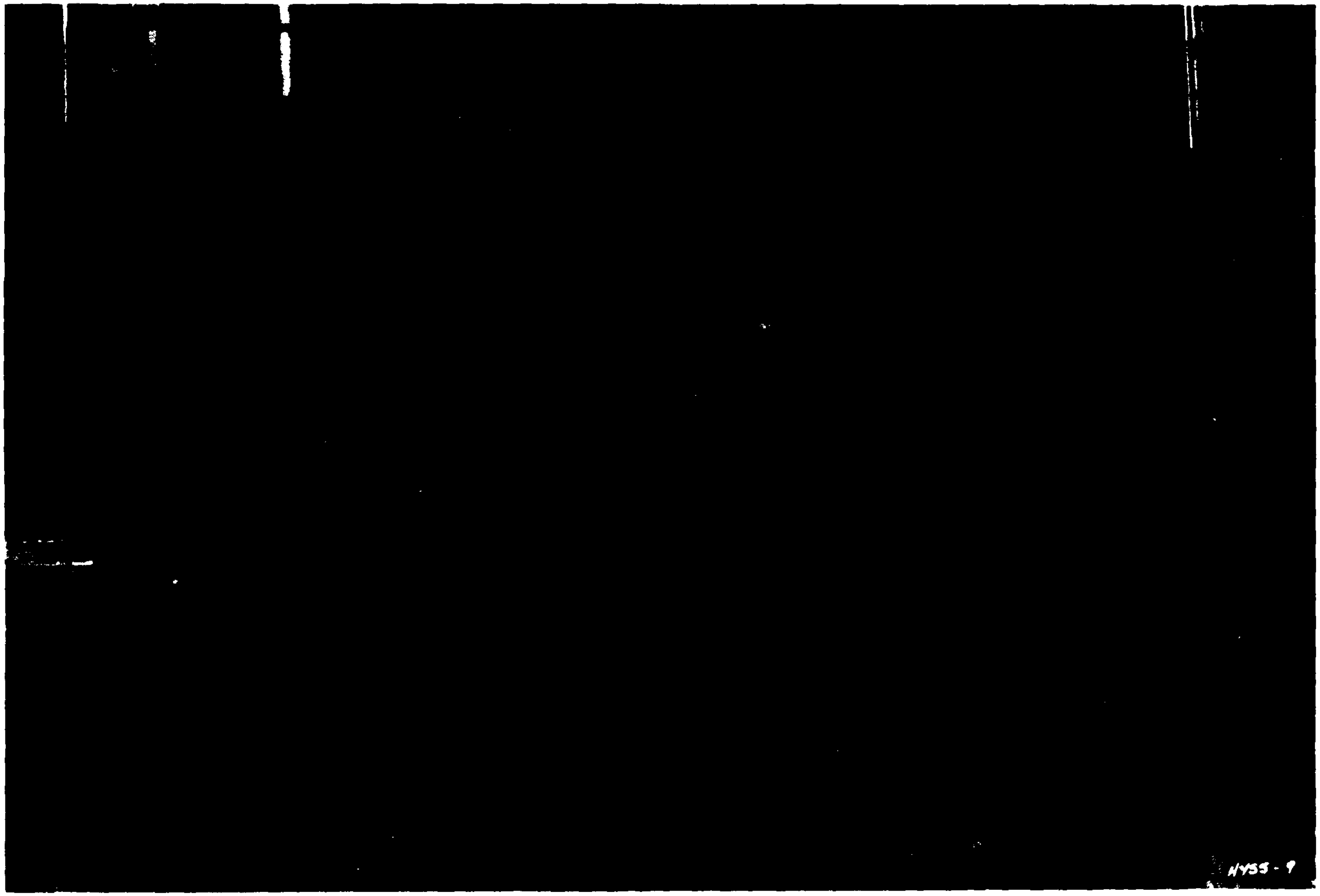


Photo 1. Intermittent vortex upstream from headwall with type 1 inlet; discharge 3,300 cfs

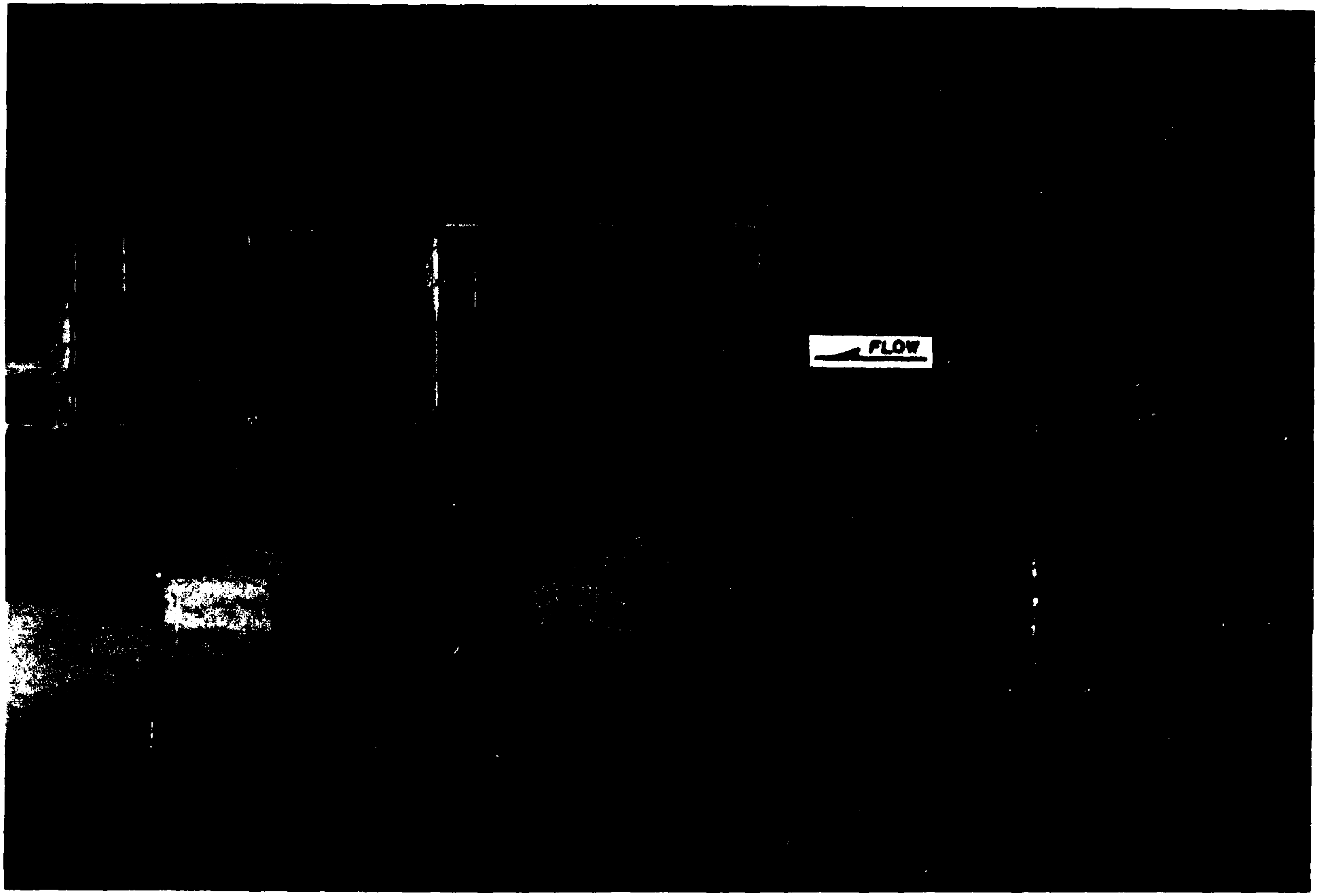
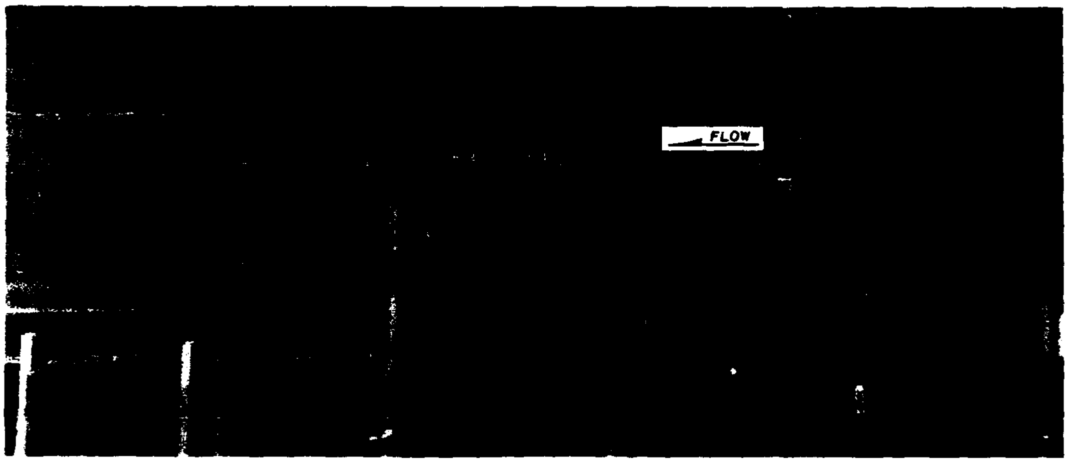


Photo 2. Side view showing flow conditions upstream of the type 2 inlet, discharge 3,300 cfs

a. Discharge 2,600 cfs

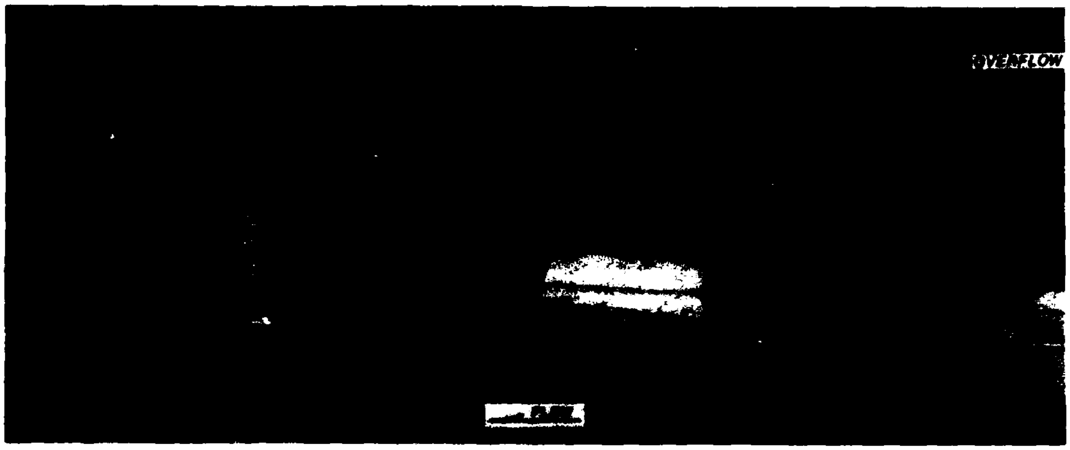
b. Discharge 2,800 cfs



c. Discharge 3,300 cfs

Photo 3. Flow conditions in transition section immediately upstream of inlet

a. Discharge 3,300 cfs



b. Discharge 3,400 cfs

c. Discharge 3,500 cfs

Photo 4. Flow conditions between sta 69+00 and 67+40 with increased wall heights installed between sta 68+45 and 67+40 (Continued)

d. Discharge 3,600 cfs

e. Discharge 4,000 cfs

Photo 4. (Concluded)

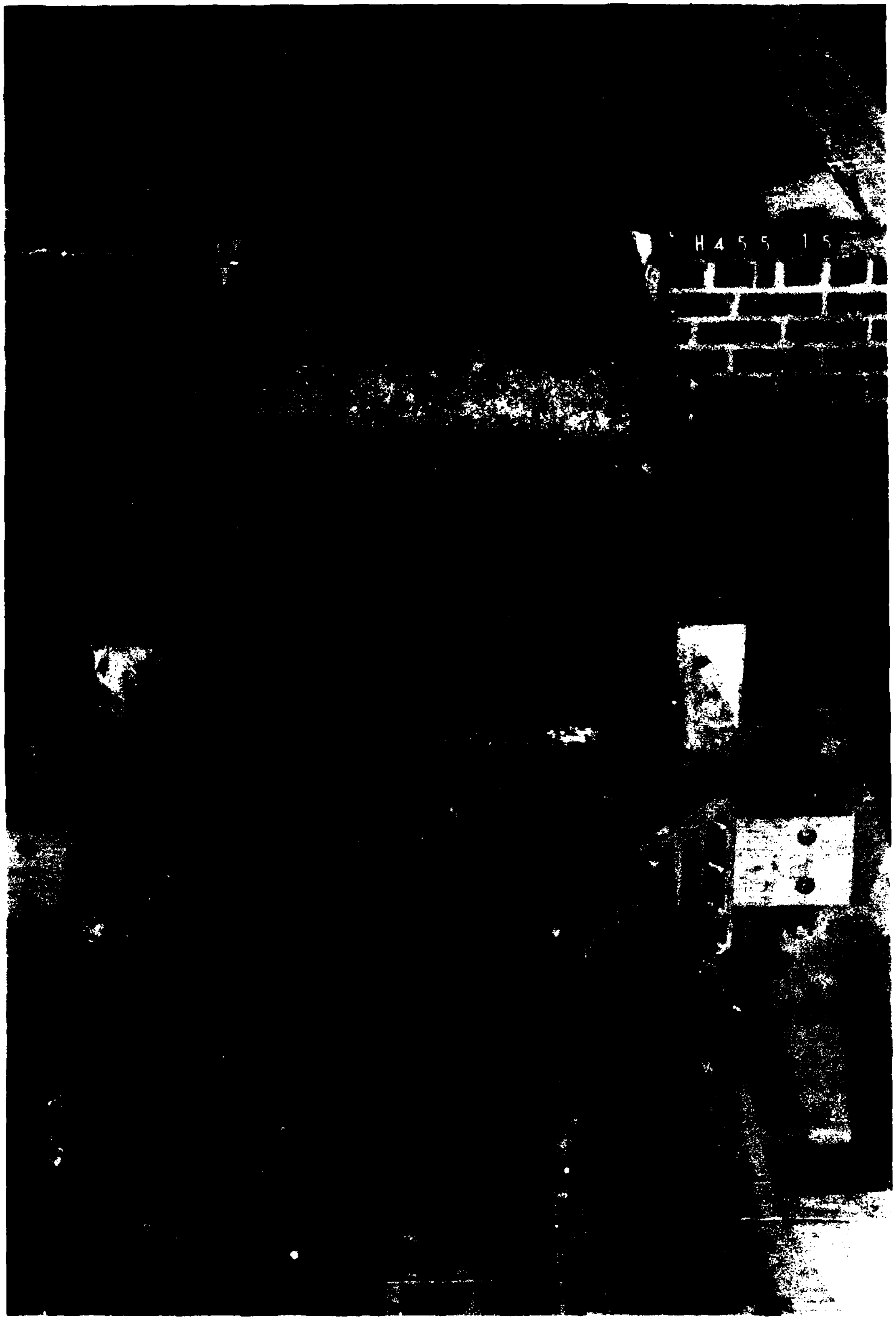
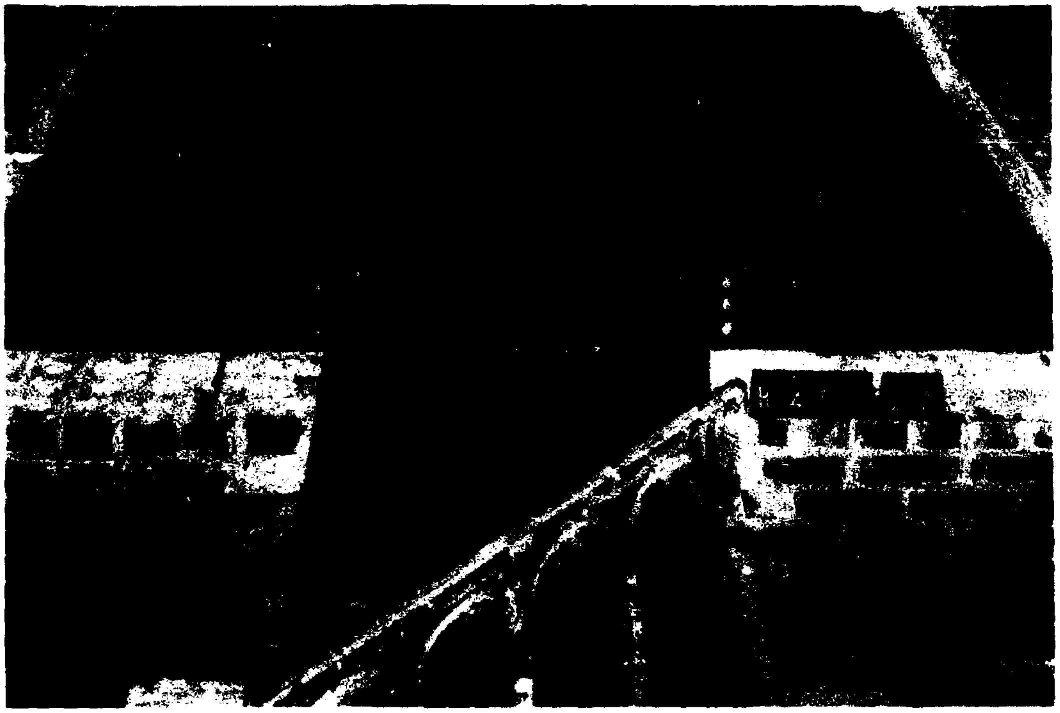
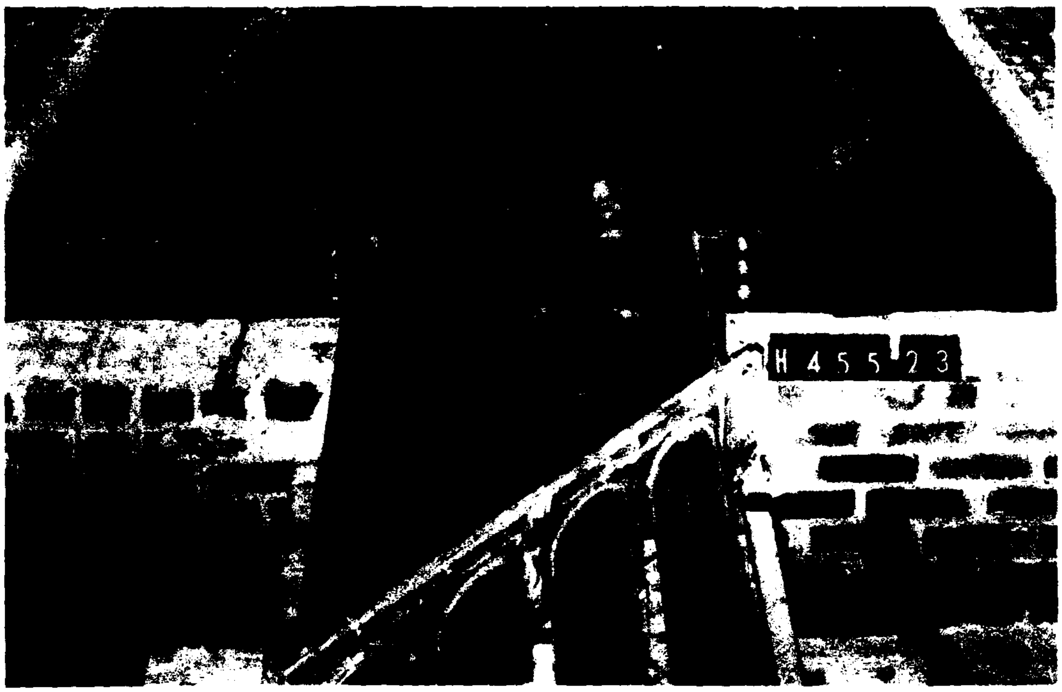


Photo 5. Flow conditions in type 1 stilling basin; discharge 3,300 cfs, tailwater el 8.85



a. Discharge 1,000 cfs, tailwater el 4.0



b. Discharge 2,000 cfs, tailwater el 6.0

Photo 6. Flow conditions in type 10 design stilling basin, looking downstream (Continued)

c. Discharge 3,300 cfs, tailwater el 8.85

Photo 6. (Concluded)

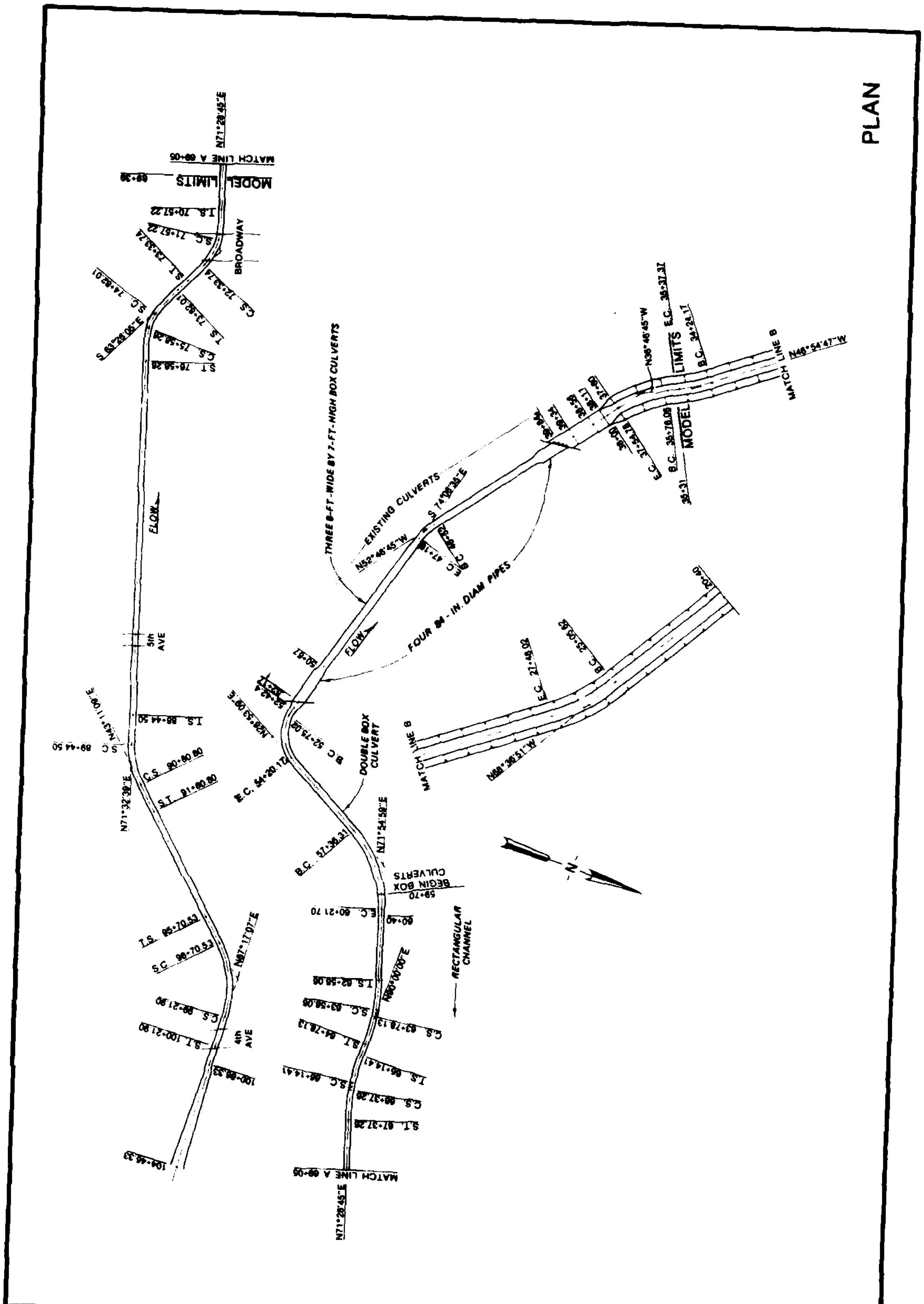
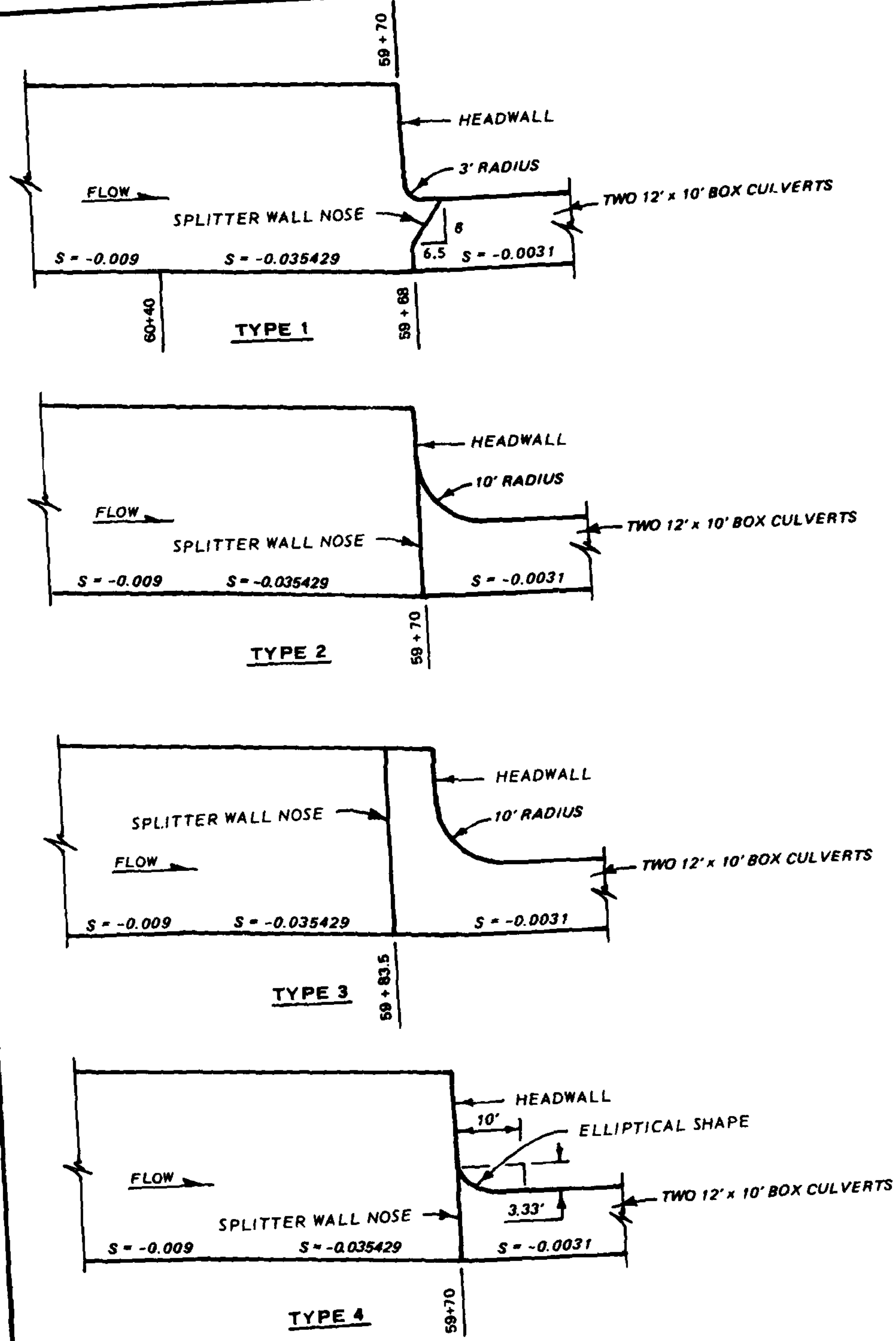

PLAN
THREE 8-FT-WIDE BY 7-FT-HIGH BOX CULVERTS
EXISTING CULVERTS
FOUR 84-IN.-DIAM PIPES
DOUBLE BOX CULVERT
BEGIN BOX CULVERTS
RECTANGULAR CHANNEL
MATCH LINE A 69+05
MATCH LINE B
MODEL LIMITS
BROADWAY
5th AVE
4th AVE
FLOW

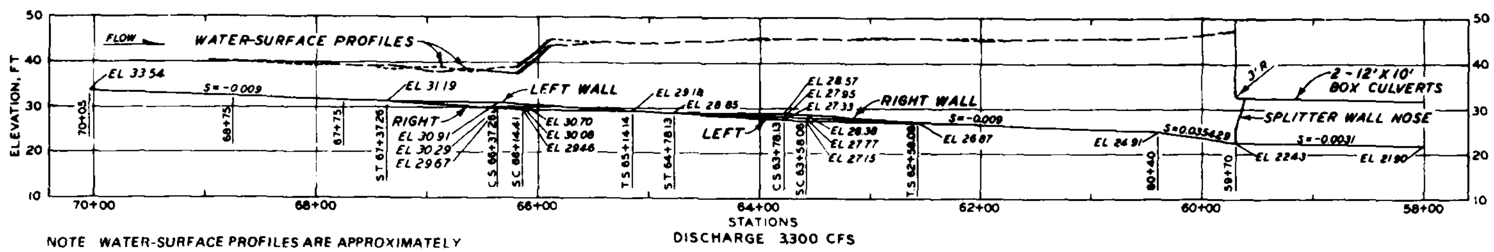
PLATE 1



INLET DESIGNS

TYPES 1 – 4

PLATE 2



NOTE WATER-SURFACE PROFILES ARE APPROXIMATELY 1.5 FT FROM WALLS.

ELEVATIONS ARE REFERRED TO THE NATIONAL GEODETIC VERTICAL DATUM

LEGEND

— — RIGHT SIDE WATER SURFACE PROFILE
- - - - LEFT SIDE WATER SURFACE PROFILE

WATER–SURFACE PROFILES
TYPE I INLET DESIGN

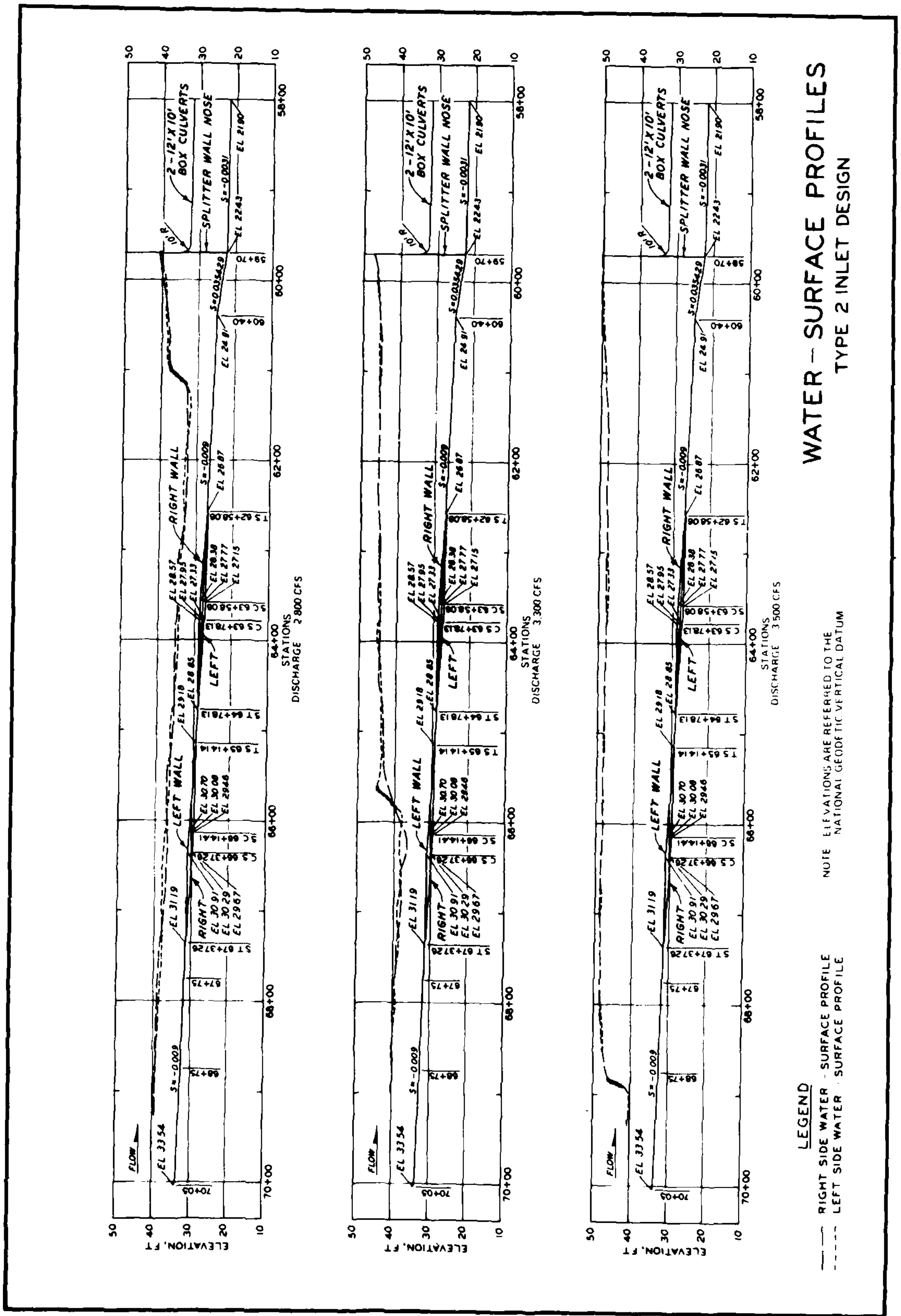

WATER - SURFACE PROFILES
TYPE 2 INLET DESIGN
LEGEND
RIGHT SIDE WATER - SURFACE PROFILE
LEFT SIDE WATER - SURFACE PROFILE
NOTE ELEVATIONS ARE REFERRED TO THE NATIONAL GEODETIC VERTICAL DATUM
STATIONS
DISCHARGE 2,800 CFS
DISCHARGE 3,300 CFS
DISCHARGE 3,500 CFS
ELEVATION, FT
FLOW
2 - 12' X 10' BOX CULVERTS
SPLITTER WALL NOSE
RIGHT WALL
LEFT WALL

PLATE 4

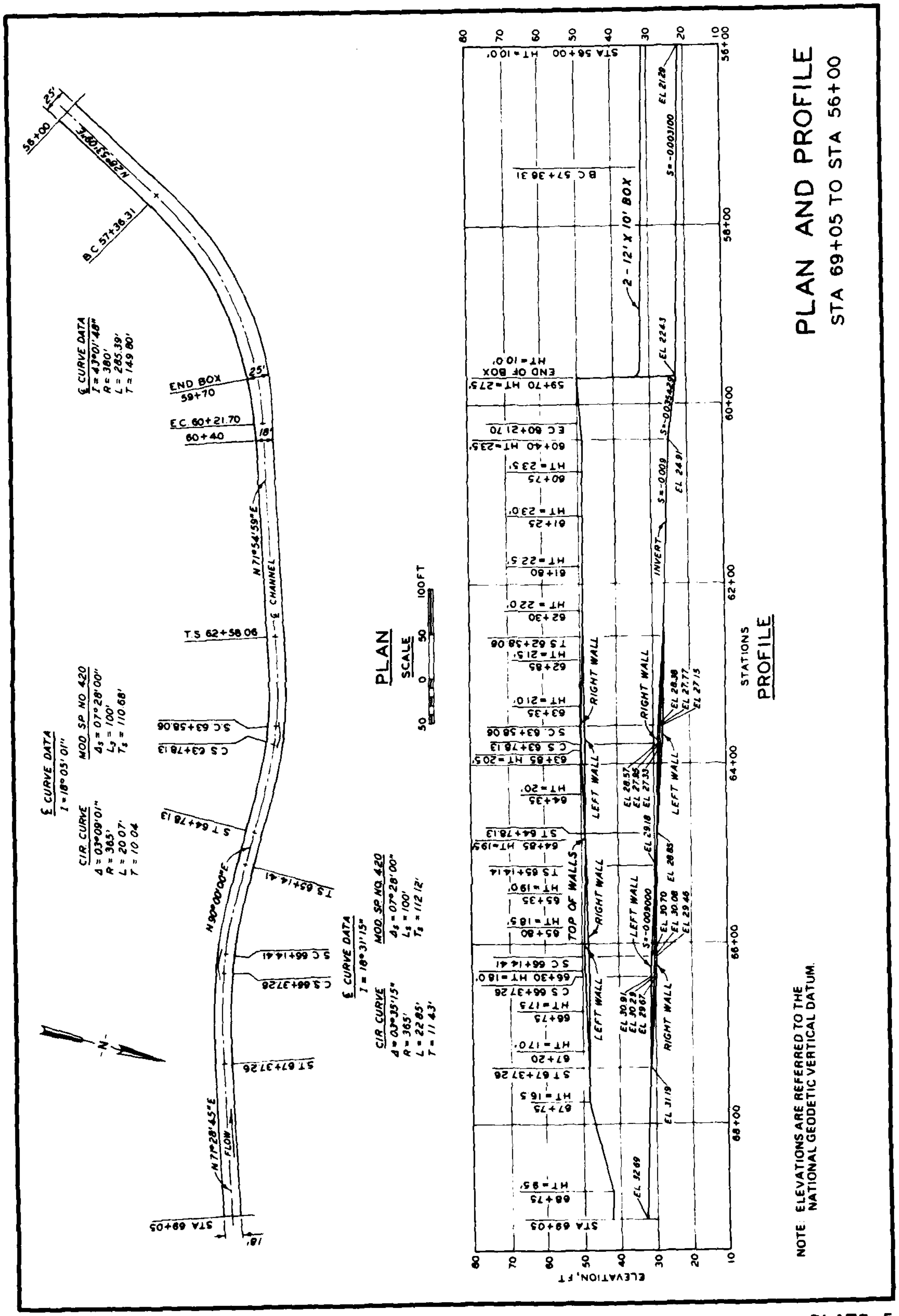

PLAN AND PROFILE
STA 69+05 TO STA 56+00
PLAN
SCALE
PROFILE
STATIONS
ELEVATION, FT
NOTE: ELEVATIONS ARE REFERRED TO THE NATIONAL GEODETIC VERTICAL DATUM.

PLATE 5

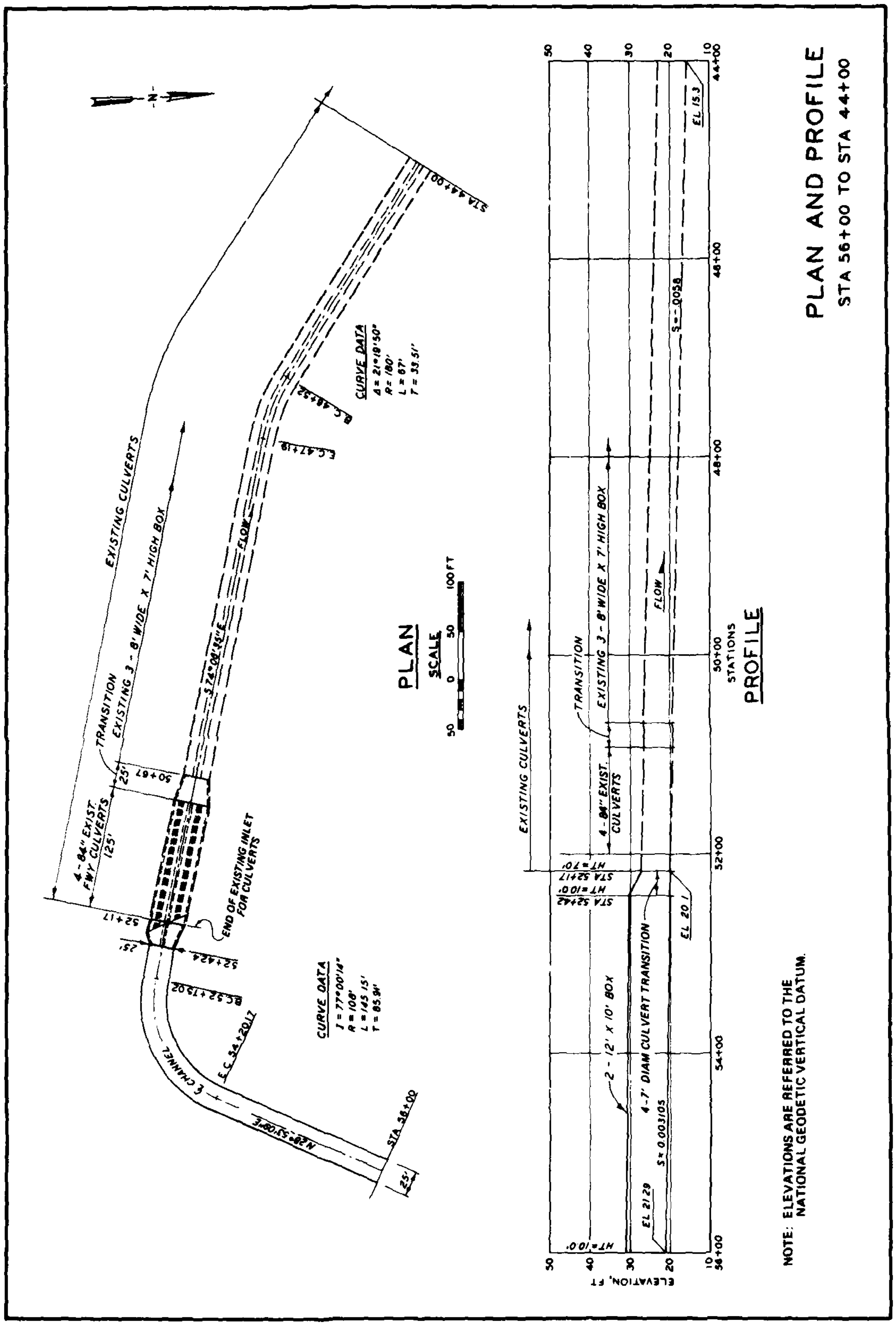
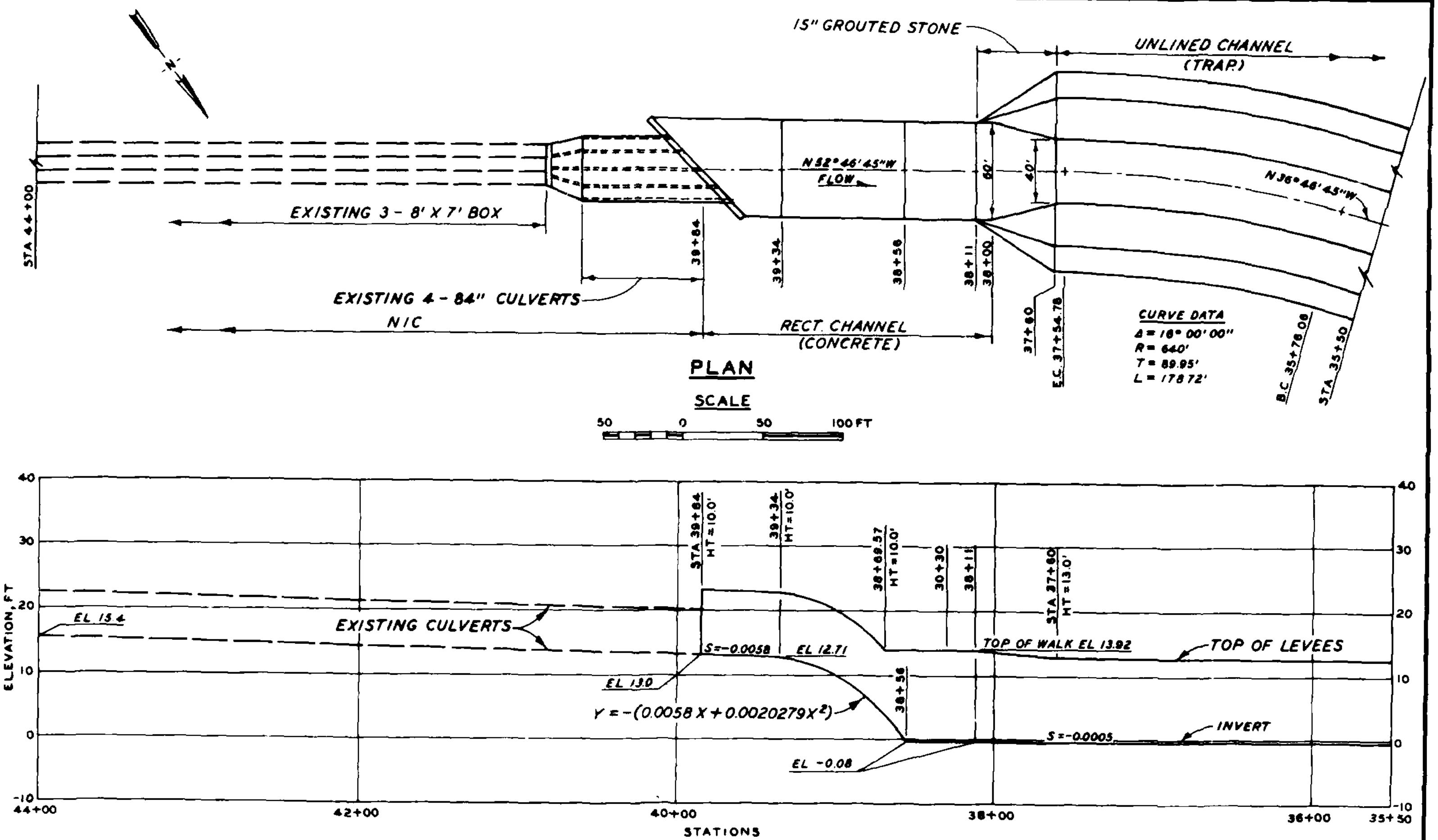

PLAN AND PROFILE
STA 56+00 TO STA 44+00
PLAN
SCALE
50 0 50 100 FT
EXISTING CULVERTS
TRANSITION
EXISTING 3 - 8' WIDE X 7' HIGH BOX
4 - 84" EXIST FWY CULVERTS
125'
25'
50+67
52+17
52+42
END OF EXISTING INLET FOR CULVERTS
S 74°08'35" E
FLOW
CURVE DATA
Δ = 21°18'50"
R = 180'
L = 67'
T = 33.51'
B.C. 48+52
E.C. 47+19
STA 44+00
CURVE DATA
I = 77°00'14"
R = 108'
L = 145.15'
T = 85.91'
B.C. 52+75.02
E.C. 54+20.17
℄ CHANNEL
N 28°53'08" E
STA 56+00
PROFILE
STATIONS
ELEVATION, FT
EXISTING CULVERTS
TRANSITION
EXISTING 3 - 8' WIDE X 7' HIGH BOX
4 - 84" EXIST. CULVERTS
FLOW
HT = 7.0'
STA 52+17
HT = 10.0'
STA 52+42
EL 20.1
2 - 12' X 10' BOX
4 - 7' DIAM CULVERT TRANSITION
S = 0.003105
EL 21.29
HT = 10.0'
S = .0058
EL 15.3
NOTE: ELEVATIONS ARE REFERRED TO THE NATIONAL GEODETIC VERTICAL DATUM.

PLATE 6

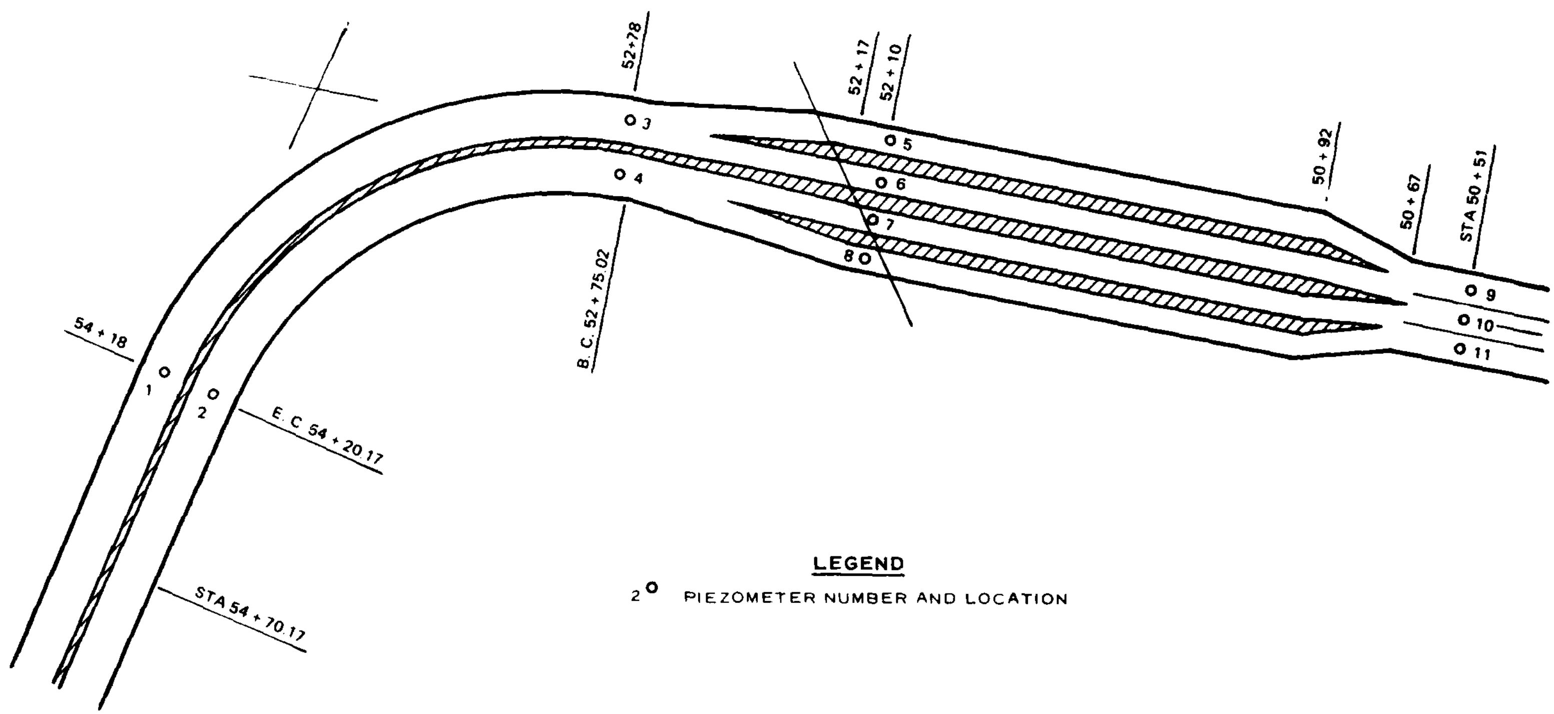


PROFILE

NOTE: ELEVATIONS ARE REFERRED TO THE NATIONAL GEODETIC VERTICAL DATUM.

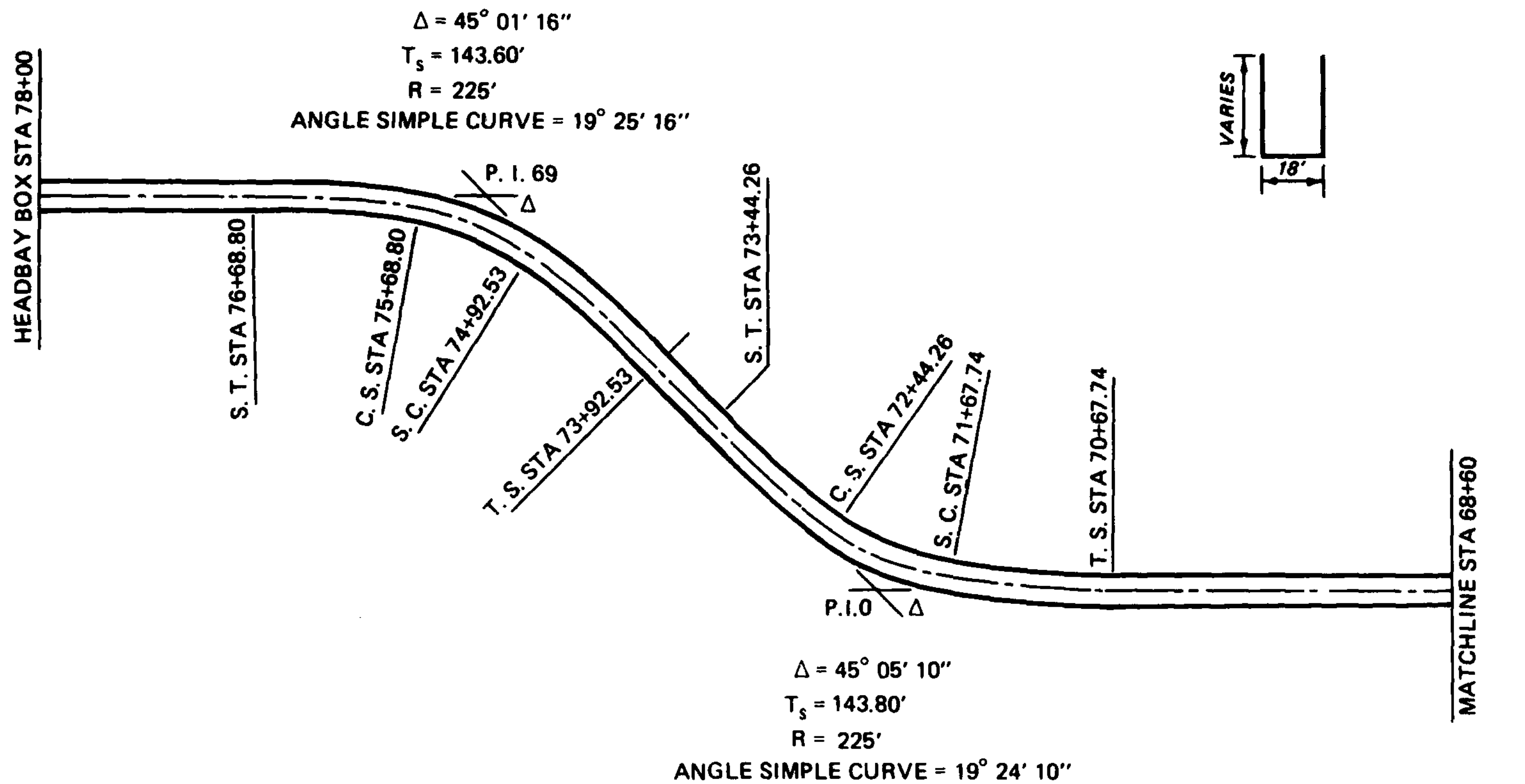
PLAN AND PROFILE

STA 44+00 TO STA 35+50



PIEZOMETER LOCATIONS

PLATE 8



PLAN VIEW
BETWEEN STA 78+00 AND STA 68+60

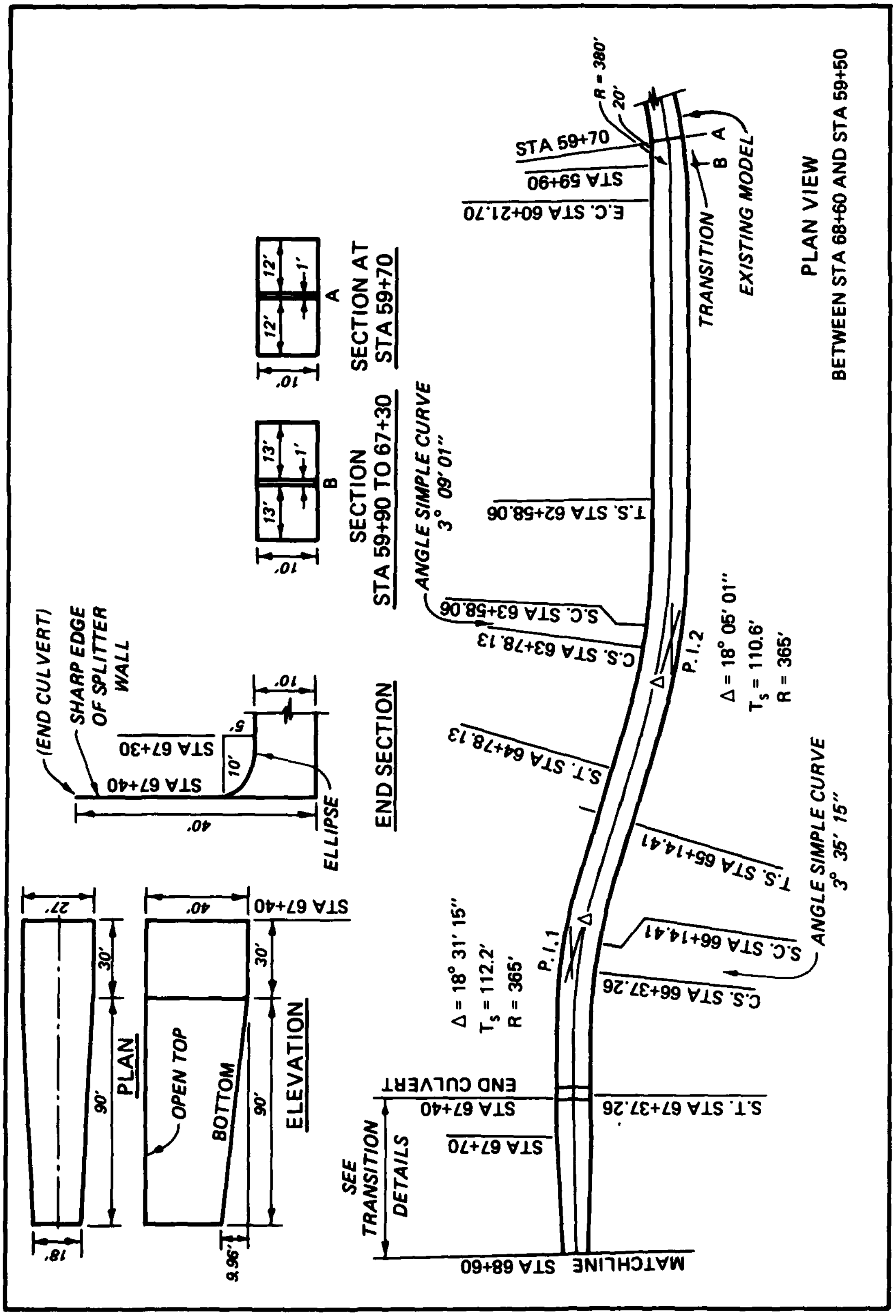
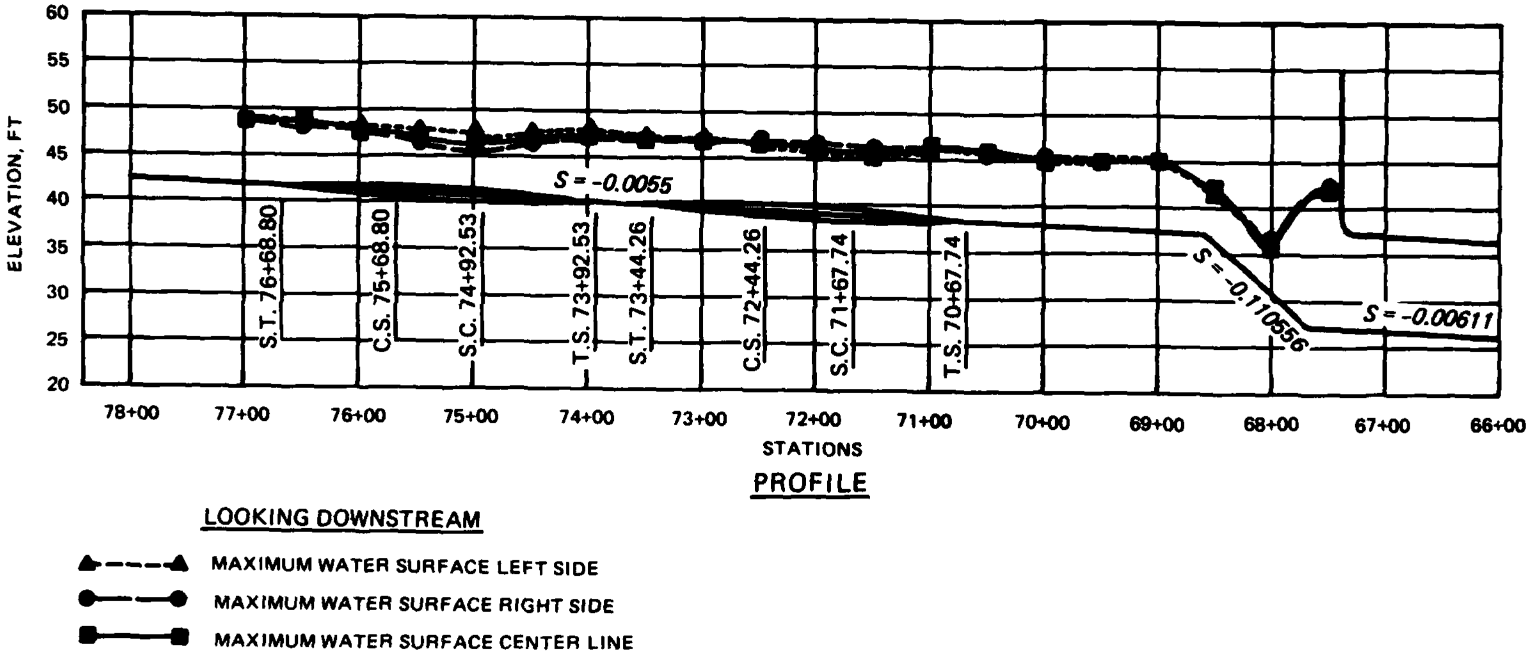

PLAN VIEW
BETWEEN STA 68+60 AND STA 59+50
R = 380'
20'
STA 59+70
STA 59+90
E.C. STA 60+21.70
A
B
TRANSITION
EXISTING MODEL
T.S. STA 62+58.06
S.C. STA 63+58.06
C.S. STA 63+78.13
P.I.2
Δ = 18° 05' 01"
Ts = 110.6'
R = 365'
S.T. STA 64+78.13
T.S. STA 65+14.41
ANGLE SIMPLE CURVE
3° 35' 15"
S.C. STA 66+14.41
C.S. STA 66+37.26
P.I.1
Δ = 18° 31' 15"
Ts = 112.2'
R = 365'
S.T. STA 67+37.26
END CULVERT
STA 67+40
STA 67+70
SEE
TRANSITION
DETAILS
MATCHLINE STA 68+60
SECTION AT
STA 59+70
12'
12'
1'
10'
SECTION
STA 59+90 TO 67+30
13'
13'
1'
10'
ANGLE SIMPLE CURVE
3° 09' 01"
END SECTION
(END CULVERT)
SHARP EDGE
OF SPLITTER
WALL
STA 67+40
STA 67+30
5'
10'
10'
40'
ELLIPSE
PLAN
27'
30'
90'
18'
ELEVATION
OPEN TOP
BOTTOM
40'
30'
90'
9.96'
STA 67+40

PLATE 10



WATER-SURFACE PROFILES
DISCHARGE 2,800 CFS
TYPE 4 DESIGN INLET

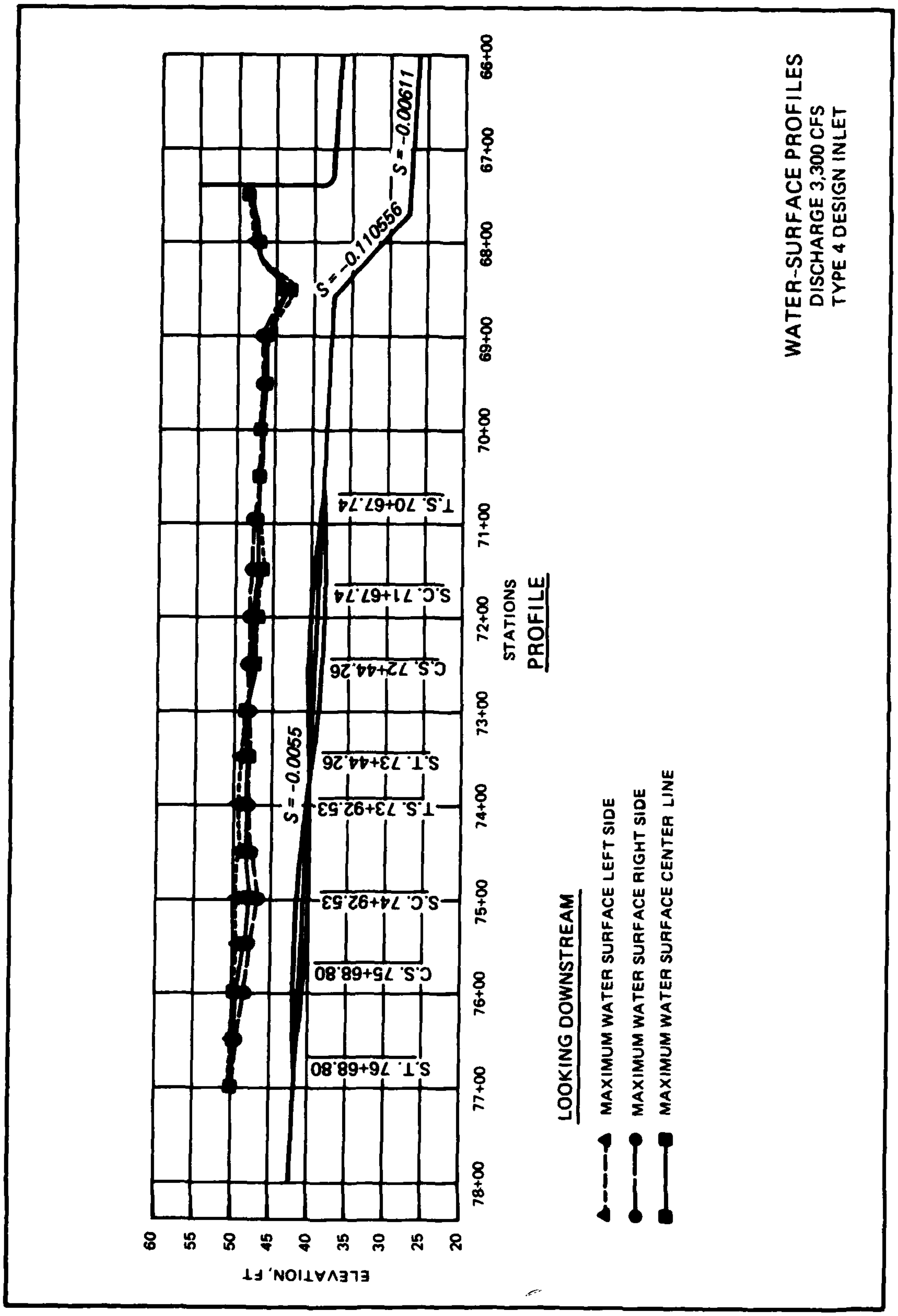
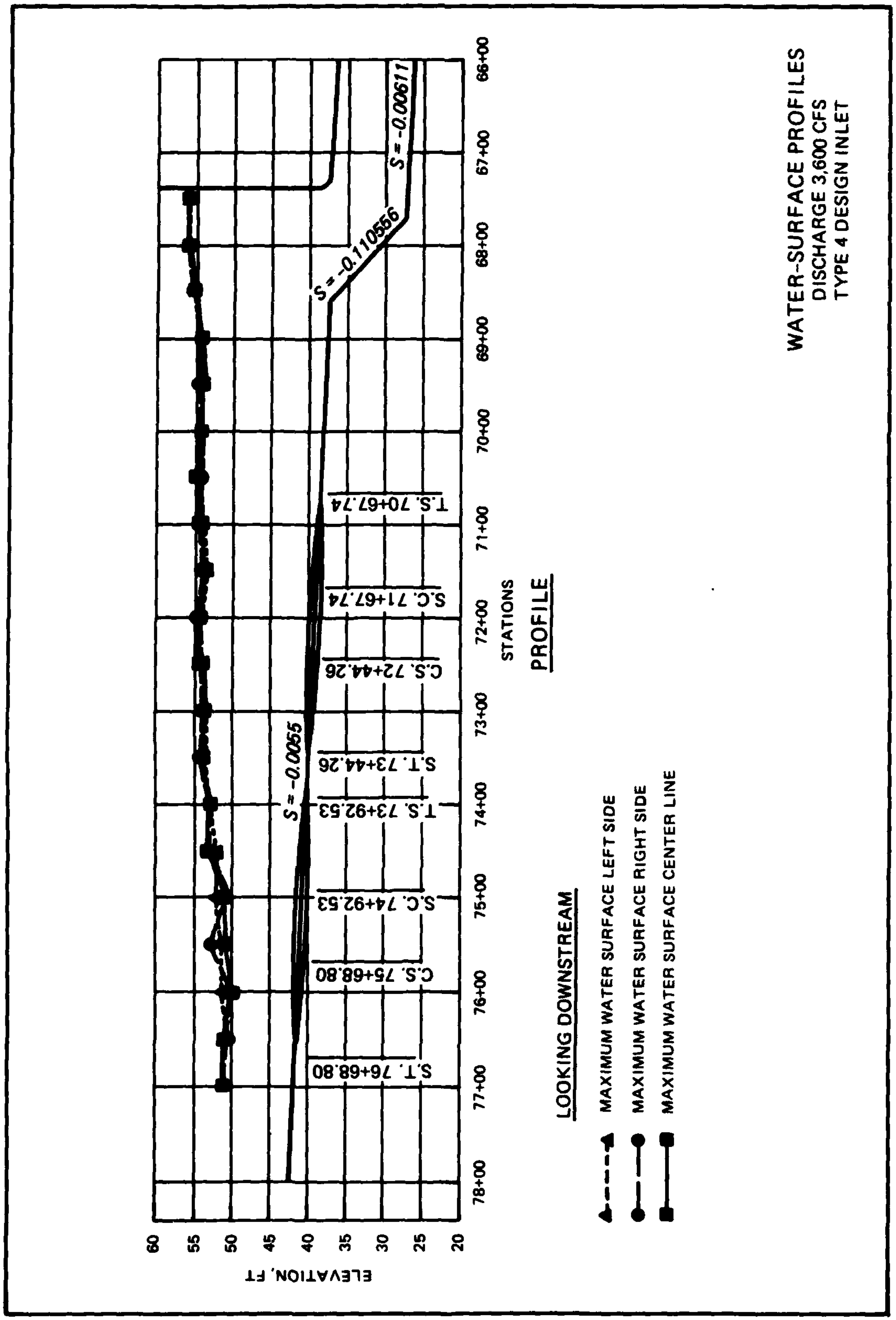

WATER-SURFACE PROFILES
DISCHARGE 3,300 CFS
TYPE 4 DESIGN INLET
PROFILE
STATIONS
ELEVATION, FT
LOOKING DOWNSTREAM
MAXIMUM WATER SURFACE LEFT SIDE
MAXIMUM WATER SURFACE RIGHT SIDE
MAXIMUM WATER SURFACE CENTER LINE
S = -0.00611
S = -0.110556
S = -0.0055
T.S. 70+67.74
S.C. 71+67.74
C.S. 72+44.26
S.T. 73+44.26
T.S. 73+92.53
S.C. 74+92.53
C.S. 75+68.80
S.T. 76+68.80
66+00
67+00
68+00
69+00
70+00
71+00
72+00
73+00
74+00
75+00
76+00
77+00
78+00
60
55
50
45
40
35
30
25
20

PLATE 12



WATER-SURFACE PROFILES
DISCHARGE 3,600 CFS
TYPE 4 DESIGN INLET
PROFILE
STATIONS
ELEVATION, FT
LOOKING DOWNSTREAM
MAXIMUM WATER SURFACE LEFT SIDE
MAXIMUM WATER SURFACE RIGHT SIDE
MAXIMUM WATER SURFACE CENTER LINE
S = -0.00611
S = -0.110556
S = -0.0055
T.S. 70+67.74
S.C. 71+67.74
C.S. 72+44.26
S.T. 73+44.26
T.S. 73+92.53
S.C. 74+92.53
C.S. 75+68.80
S.T. 76+68.80
66+00
67+00
68+00
69+00
70+00
71+00
72+00
73+00
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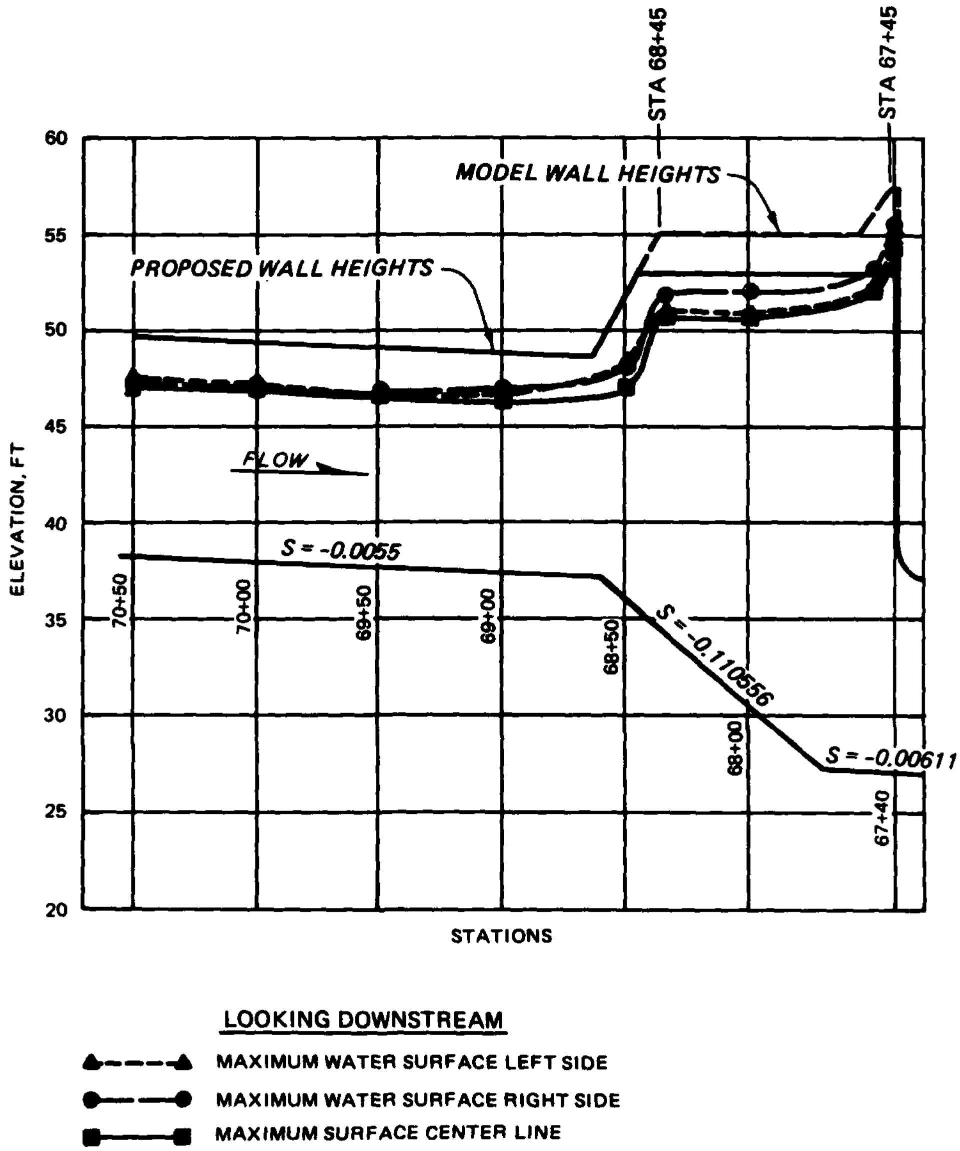
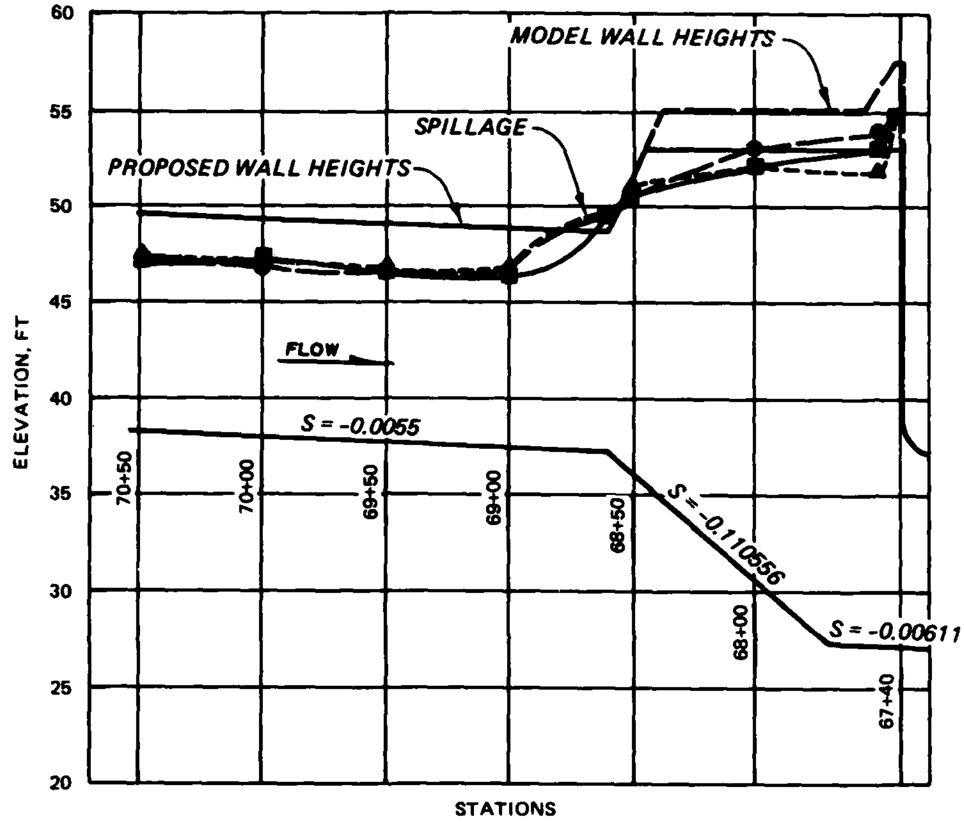
40
35
30
25
20



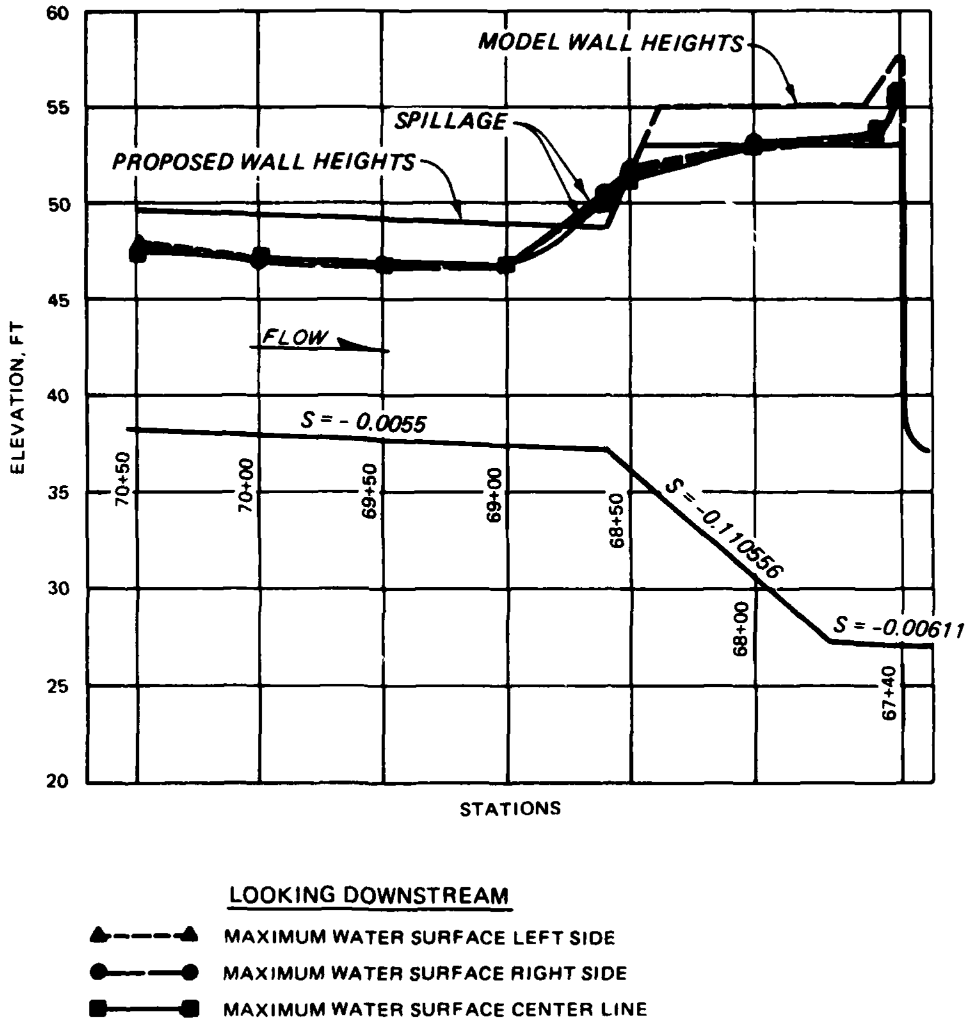
WATER-SURFACE PROFILES
DISCHARGE 3,300 CFS
INCREASED WALL HEIGHTS



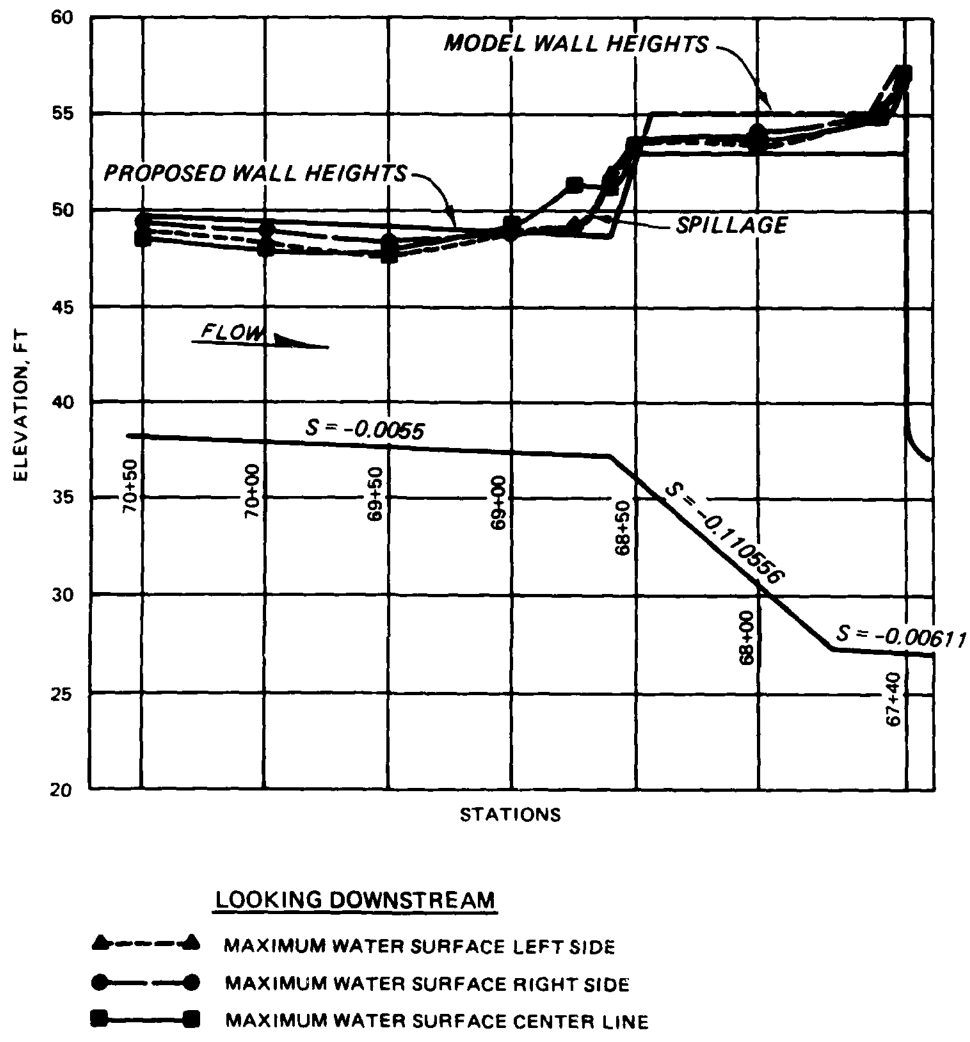
LOOKING DOWNSTREAM

- ▲- - - -▲ MAXIMUM WATER SURFACE LEFT SIDE
- ●— — —● MAXIMUM WATER SURFACE RIGHT SIDE
- ■———■ MAXIMUM WATER SURFACE CENTER LINE

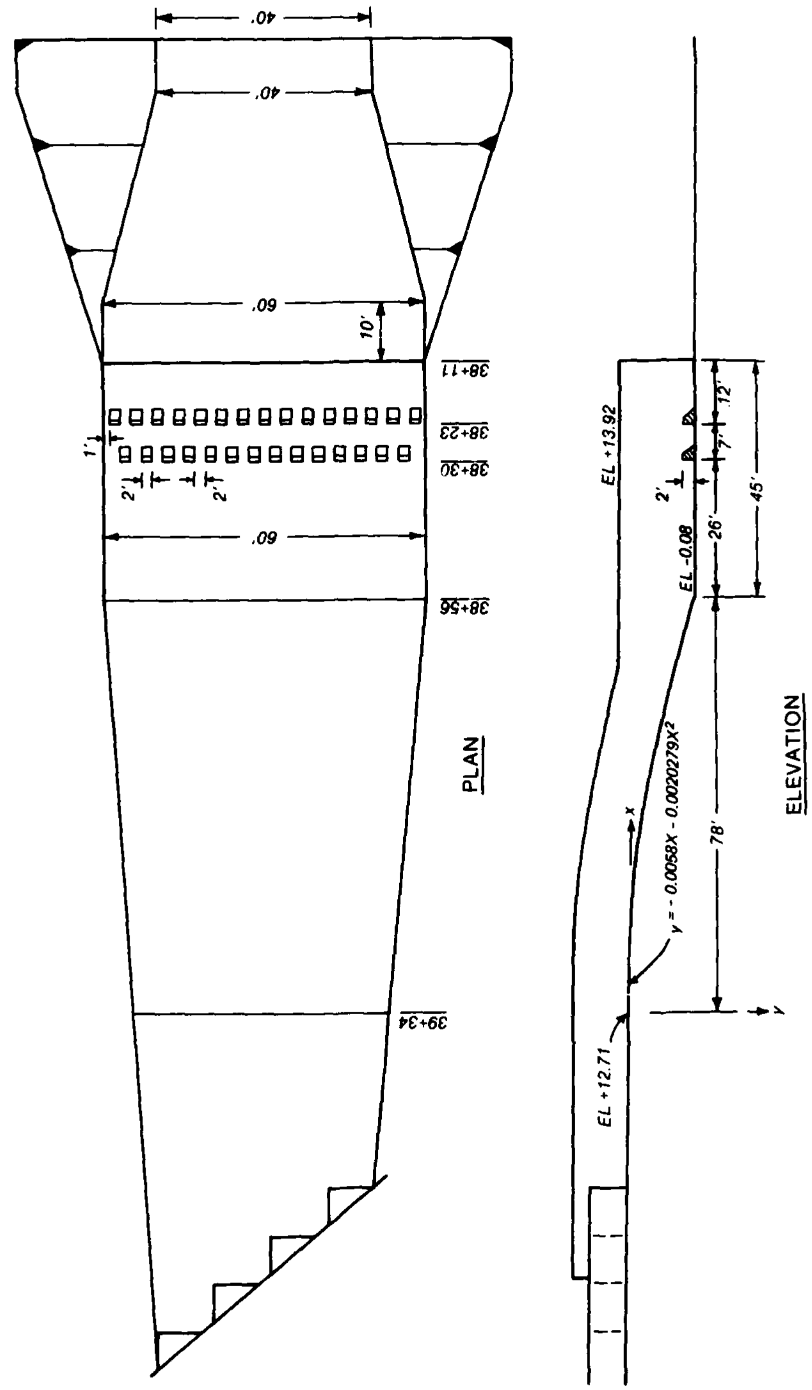
WATER-SURFACE PROFILES
DISCHARGE 3,400 CFS
INCREASED WALL HEIGHTS



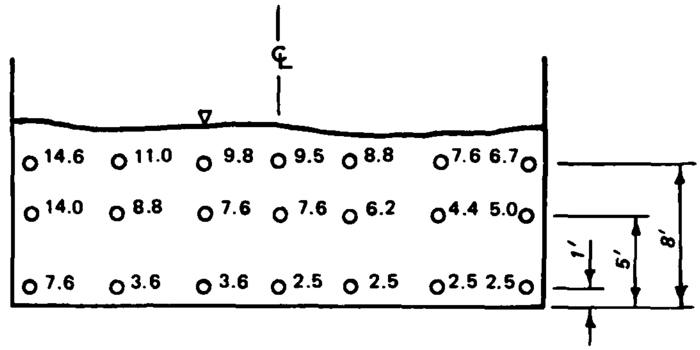
WATER-SURFACE PROFILES
DISCHARGE 3,500 CFS
INCREASED WALL HEIGHTS



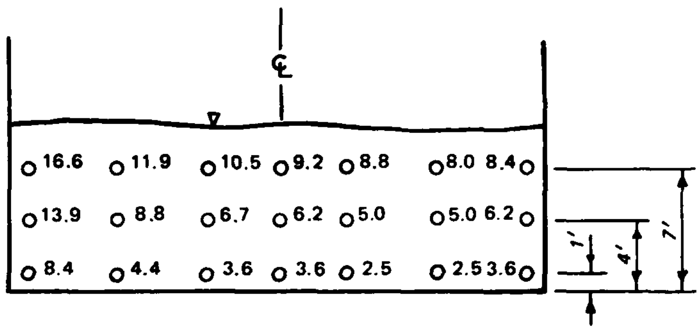
WATER-SURFACE PROFILES
DISCHARGE 4,000 CFS
INCREASED WALL HEIGHTS



TYPE 1 (ORIGINAL) STILLING BASIN



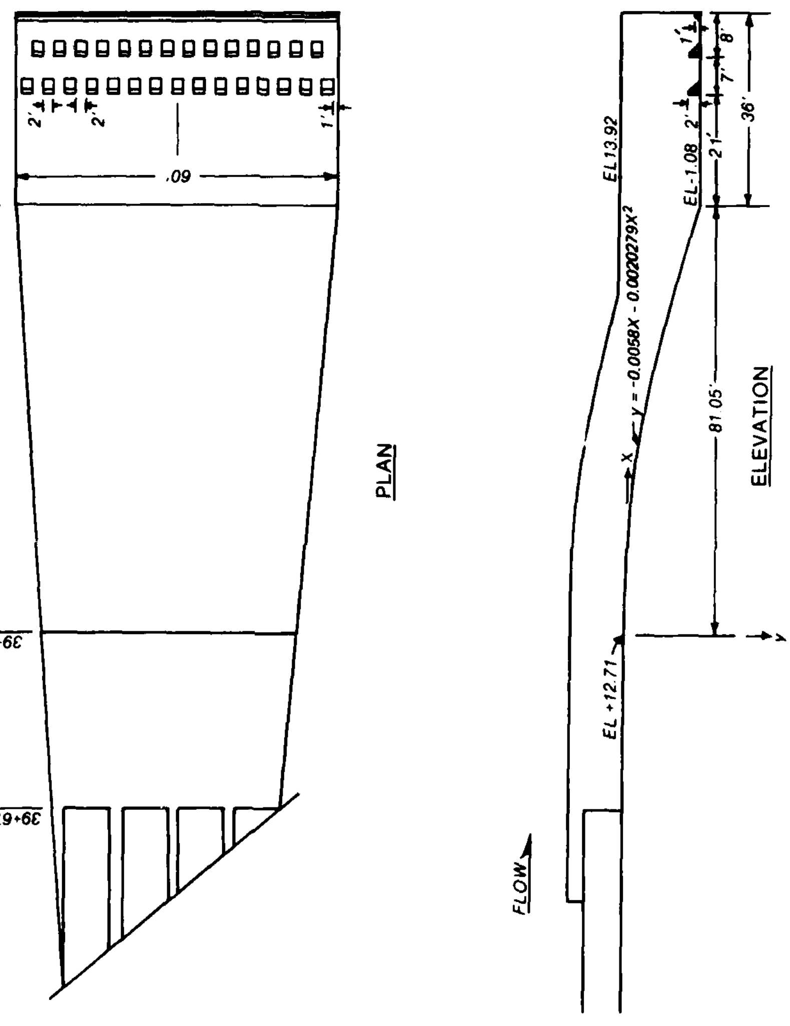
TAILWATER EL 9.85



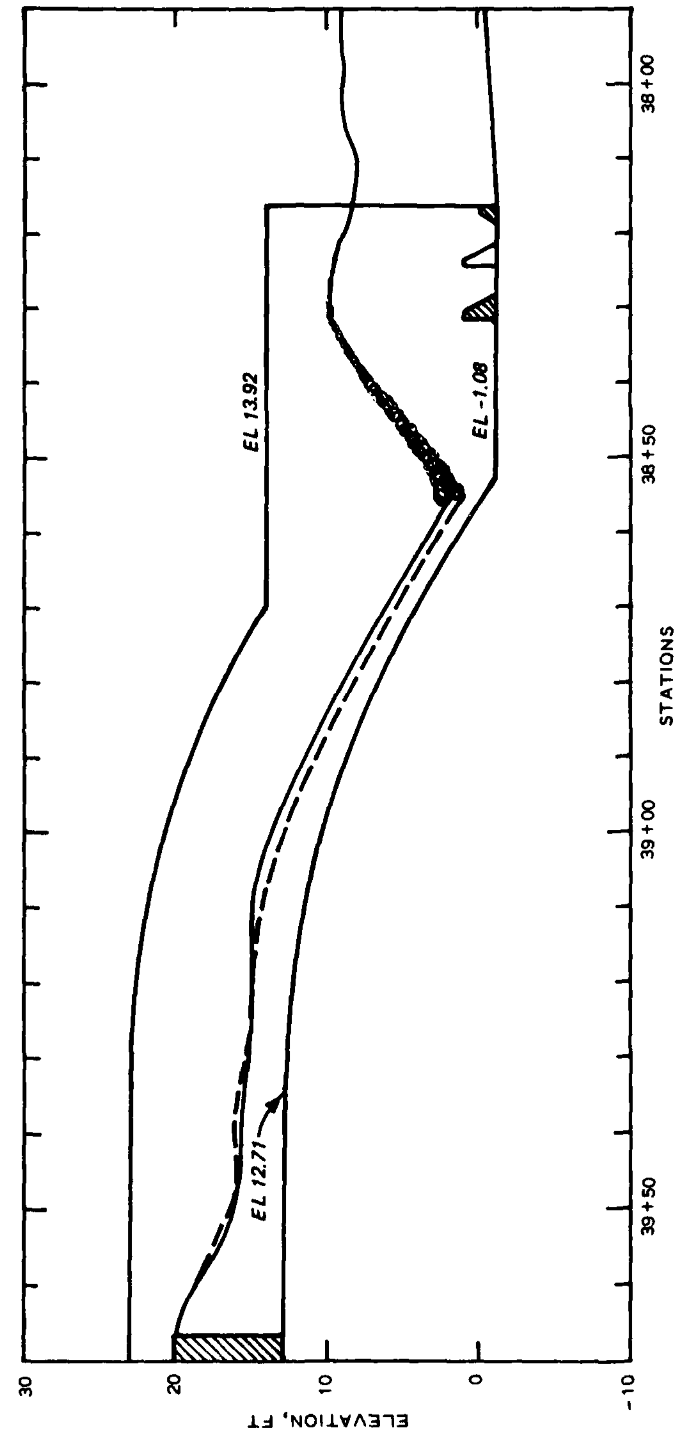
TAILWATER EL 8.85

NOTE: SECTIONS SHOWN LOOKING DOWNSTREAM
VELOCITIES GIVEN IN FEET PER SECOND

MAXIMUM VELOCITIES
STATION 38+11
TYPE 1 STILLING BASIN
DISCHARGE 3,300 CFS



TYPE 2 STILLING BASIN

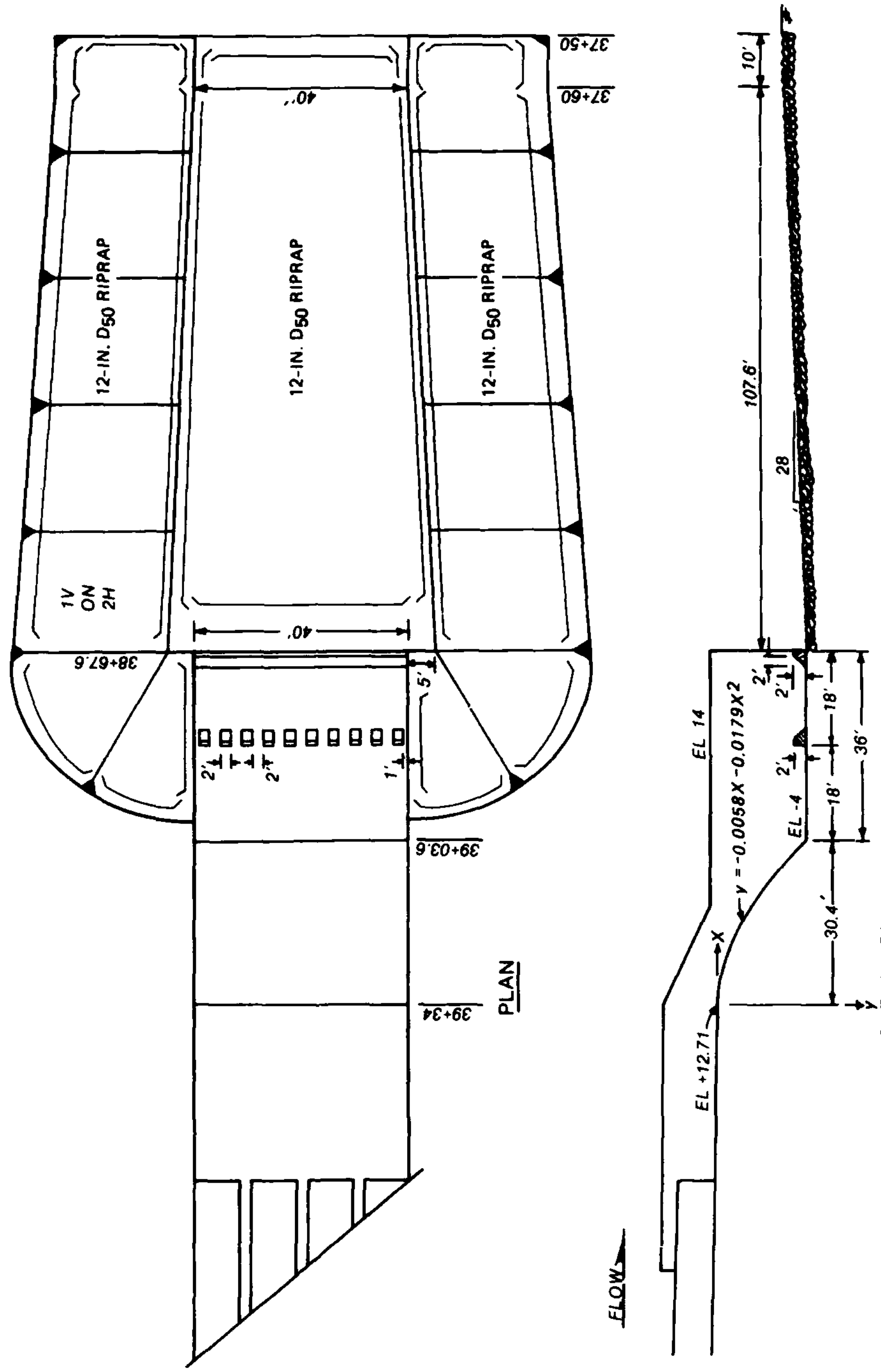


LEGEND

—— LEFT SIDE LOOKING DOWNSTREAM

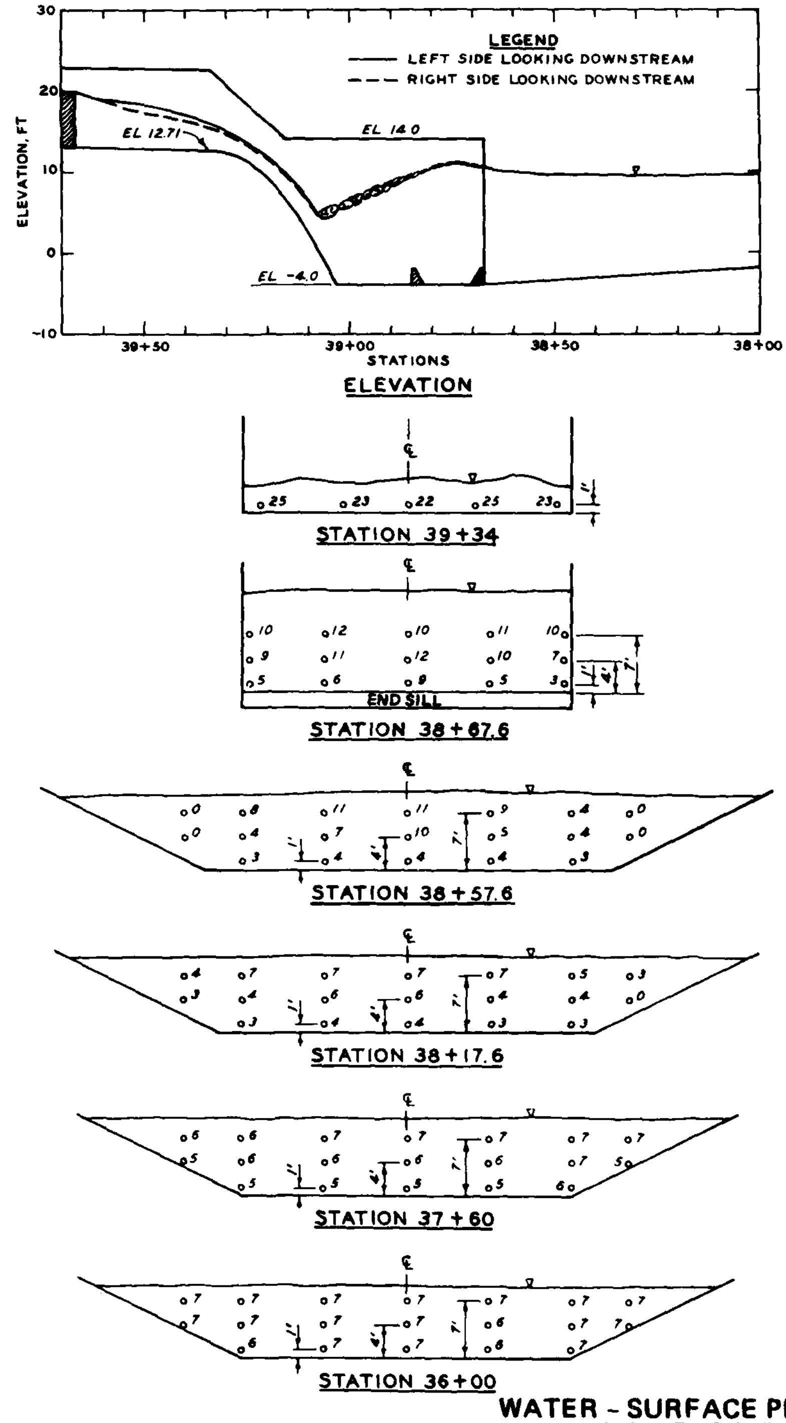
— — — RIGHT SIDE LOOKING DOWNSTREAM

WATER-SURFACE PROFILES
TYPE 2 STILLING BASIN
DISCHARGE 3,300 CFS
TAILWATER EL 8.85



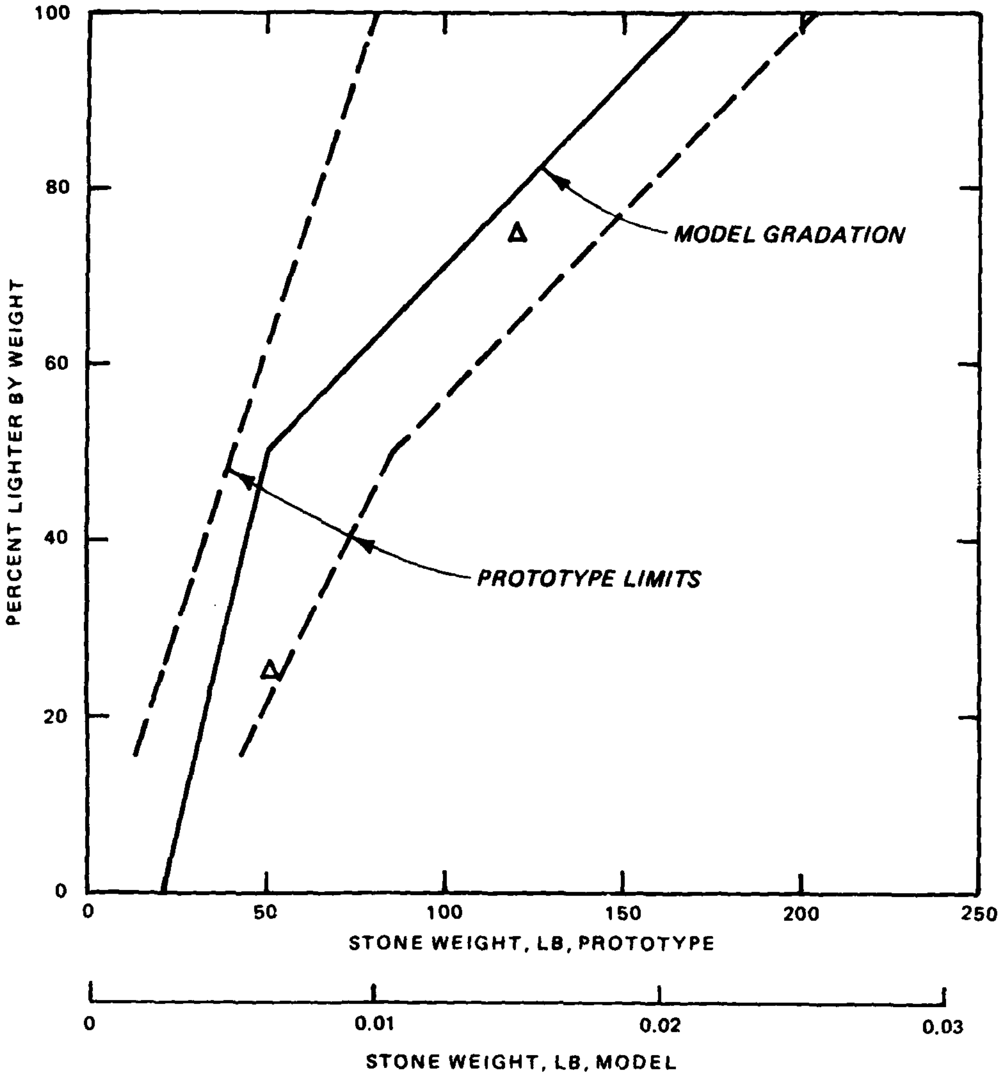
TYPE 10 STILLING BASIN
TYPE 1 RIPRAP PLAN

NOTE: RIPRAP BLANKET THICKNESS = 24 IN.



NOTE: SECTIONS SHOWN LOOKING DOWNSTREAM

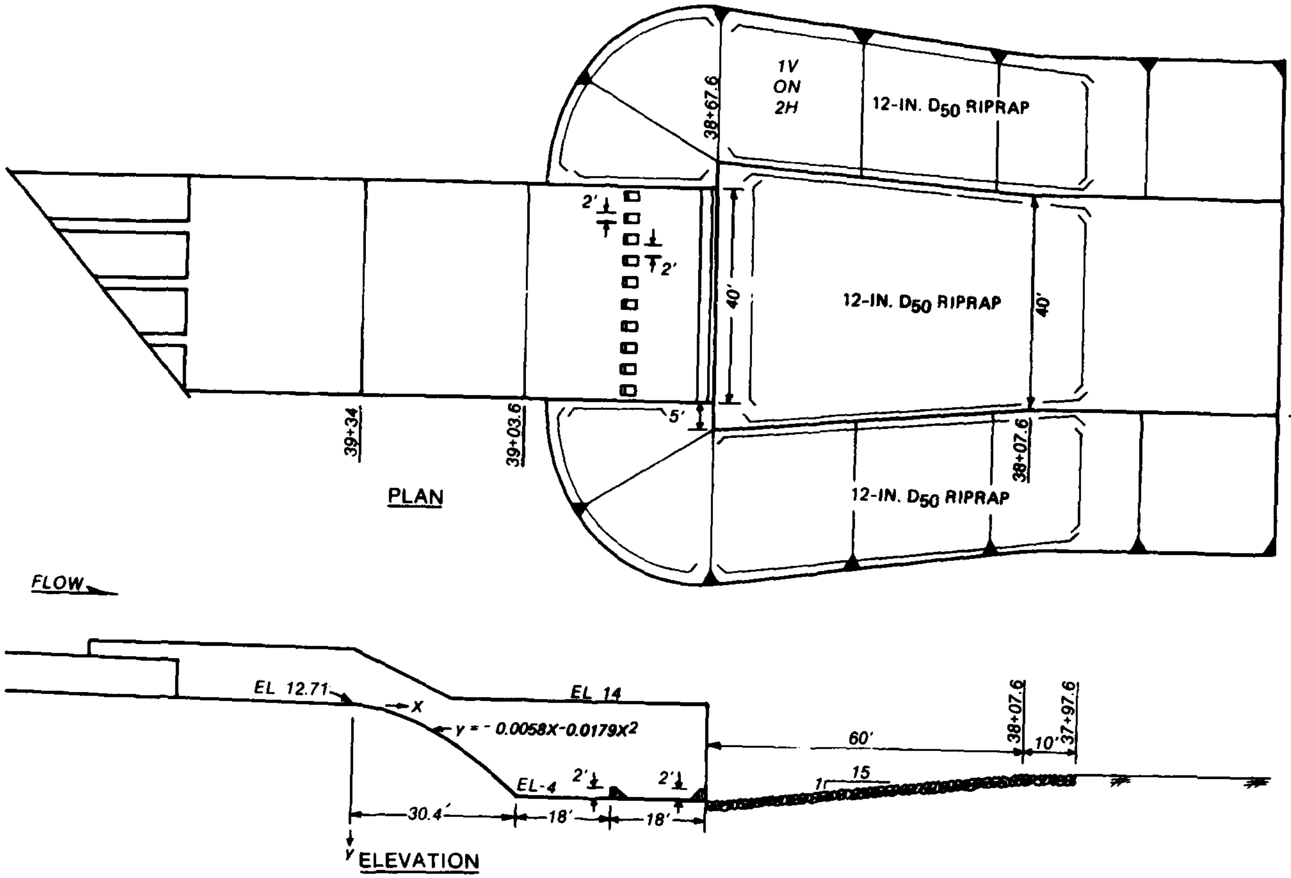
WATER - SURFACE PROFILES AND VELOCITIES
TYPE 10 STILLING BASIN
TYPE 1 DESIGN RIPRAP PLAN
DISCHARGE 3,300 CFS
TAILWATER EL 8.85



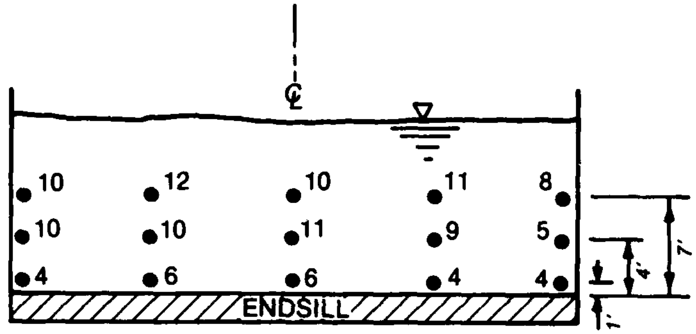
LEGEND

Δ AVERAGE WEIGHT OF STONES IN INDIVIDUAL MODEL SIEVE GRADATION

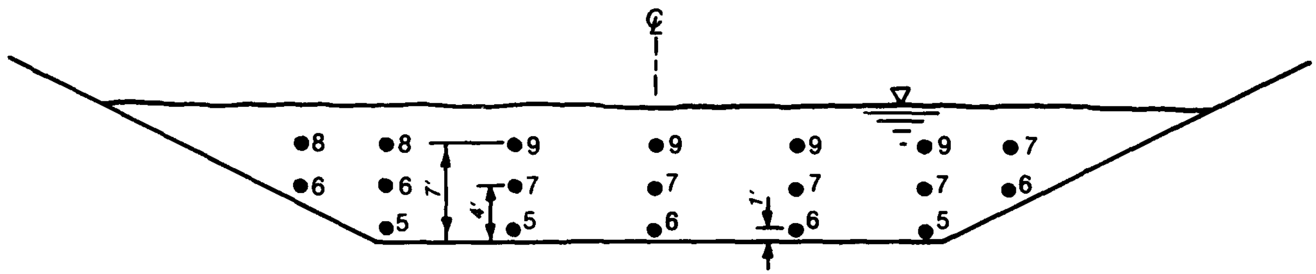
RIPRAP GRADATION
12-IN. D_{50} RIPRAP
BLANKET THICKNESS 24 IN.



TYPE 10 STILLING BASIN
TYPE 2 RIPRAP PLAN



STA. 38 + 67.6



STA. 38 + 07.6

NOTE: SECTIONS SHOWN LOOKING DOWNSTREAM
VELOCITIES GIVEN IN FEET PER SECOND

MAXIMUM VELOCITIES
TYPE 10 DESIGN STILLING BASIN
TYPE 2 DESIGN RIPRAP PLAN
DISCHARGE = 3,300 CFS
TAILWATER EL = 8.85